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Pre-Excavation Grouting

in Rock Tunneling

Expanding Horizons

Underground

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«Photo: courtesy of AF Spesialprosjekt, Tunnel Lofast, Norway».

Pre-Excavation Grouting in Tunneling

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1. INTRODUCTION

1.1. Reasons for grouting in tunneling

Tunnel excavation involves a certain risk of unexpected ground conditions. One of the risks is the chance of hitting large quantities of high pressure ground water. Smaller volumes of ground water ingress can also cause problems in a tunnel or its surroundings. Water is the most frequent reason for grouting the rock that surrounds tunnels. Ground water ingress can be controlled or handled by drainage, pre-excavation grouting and post-excavation grouting.

Rock or soil conditions causing stability problems for tunnel excavation is another possible reason for grouting. Poor and unstable ground can be improved by filling discontinuities with grout material which has sufficient strength and adhesion.

1.2. Short explanation of the subject

Pressure grouting in rock is executed by drilling boreholes of a suitable diameter, length and direction into the bedrock, placing packers near the borehole opening (or using some other means of providing a pressure tight connection to the borehole), connecting a grout conveying hose or pipe between a pump and the packer, and pumping prepared grout by overpressure into the cracks and joints of the surrounding rock.

In tunnel grouting there are two fundamentally different situations to be aware of:

- Pre-excavation grouting, or pre-grouting, where the boreholes are drilled from the tunnel excavation face into virgin rock in front of the face. The grout is pumped in and allowed to set before advancing along the tunnel face through the injected and sealed rock volume. Sometimes, such pre-excavation grouting can be executed from the ground surface, primarily for shallow tunnels with free access to the ground surface area above the tunnel.
- Post-excavation grouting, or post-grouting, is where the drilling for grout holes and pumping in of the grout material takes place somewhere along the already excavated part of the tunnel. Such locations

are usually selected where unacceptable amounts of water ingress occur. See Figure 1.1.

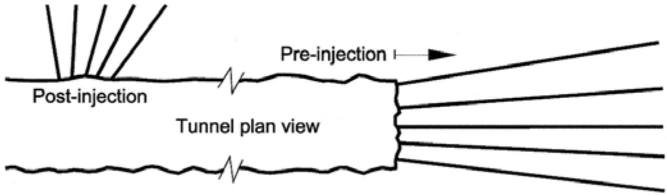


Figure 1.1 Pre-excavation grouting and post-grouting

The purpose of tunnel grouting in the majority of cases is ground water ingress control. Improvement of ground stability may sometimes be the main purpose, but will more often be a valued secondary effect of grouting for ground water control. Cement based grouts are used more often than any other grout material in tunnel injection, but there are also a number of effective chemical grouts and mineral grouts available.

Pressure grouting (injection) into the rock mass surrounding a tunnel is a technique that has existed for more than 60 years, and it has developed rapidly during the last 20 years. An important part of developing this technology into a high-efficiency economic procedure has taken place in Scandinavia. Pressure injection has been successfully carried out in a range of rock formations, from weak sedimentary rocks to granitic gneisses, and has been used against very high hydrostatic head (up to 500 m water head), as well as in shallow urban tunnels with low water head.

The result of correctly carrying out pre-grouting works ranges from drip free tunnels (less than 1.0 l/min per 100 m tunnel, [1.1] and [10.4]), to ground water ingress reduction that only takes care of the larger ingress channels. As an example, sub-sea road tunnels in Norway are mainly targeting an ingress rate of about 30 l/min per 100 m tunnel, as this produces a good balance between injection costs and lifetime dewatering costs [1.2].

Special NOTE:
It must already be emphasized at this stage that post-grouting alone cannot achieve any of these results unless very high costs are accepted.

Due to this, post-grouting should only be considered a supplementary method to pre-injection, and this important aspect of tunnel grouting will be explained later in this chapter.

1.3. Scope of the book

The primary scope of this book is pressure grouting around tunnels in rock, excavated by drill and blast, or by mechanical excavation, using cement based grouts and suitable supplementary chemical and mineral grouts. Foundation grouting and grouting in soil will be presented as well, but to a lesser degree. There are several textbooks available that cover this subject in depth.

The latest technical developments are linked to improvements in material technology, but also better equipment and improved practical procedures. The aim of this book is to provide a guide on how carry out the procedures, based on state of the art techniques. To explain why and how things have changed compared to traditional techniques, they are described to illustrate the advantages of the new methods. The presented material technology is based on BASF's MEYCO product range. BASF is the only supplier which offers products from all four injection product groups (colloidal silica, cement, PU and acrylic).

The book presents practical application techniques of pressure grouting ahead of the tunnel or shaft face and around already excavated tunnel sections. The practical focus is supported by theory when this is found to be appropriate and a contribution to explaining real life observations. Practical experience and case studies are therefore extensively used and complex theory is deliberately avoided.

When looking for available literature about grouting, the somewhat arbitrary feeling is that 90% if not more, is dealing with grouting in soil, foundation grouting and a range of post-grouting techniques for repair and water ingress control. The very important advantages of pre-grouting, when this is a possible option, have received very little attention and are almost completely undocumented in professional literature. This book is an attempt to fill some of this information gap.

1.4. Traditional cement based grouting technology

Pressure grouting into rock was initially developed for hydropower dam foundations and for general ground stabilization purposes. For such works there are normally very few practical constraints on the available working area. As a result, grouting was mostly a separate task and could be carried out without affecting or being affected by other site activities.

The traditional cement injection techniques were therefore applicable without too many disadvantages. The characteristic method of execution was:

- Figure 1.2 demonstrates the extensive use of Water Pressure Testing (WPT) on short sections of boreholes (3–5m) for mapping of the water conductivity along the length of the hole. This process involves carrying out water pressure tests at regular intervals along the borehole to see what the overall water loss situation is, i.e. which sections of the borehole are watertight and which sections allow more or less water to escape. The results were used for decision making regarding cement suspension mix design such as water/cement ratio (w/c-ratio by weight) and to choose between using cement or other grouts.

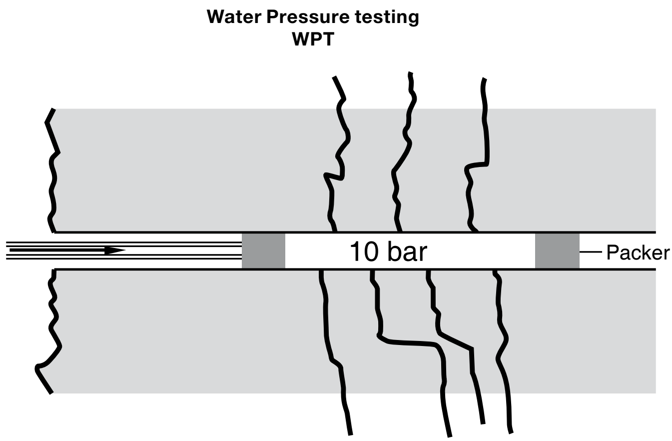
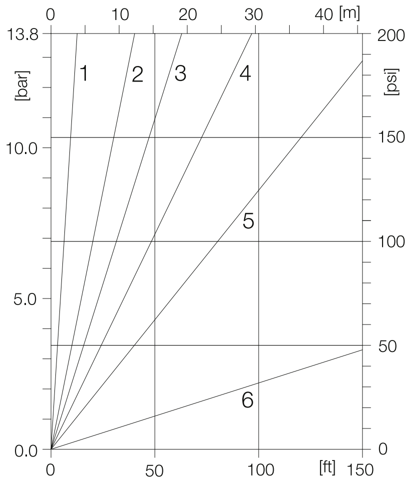


Figure 1.2 Water pressure testing of borehole

- Use of variable and mostly very high w/c-ratio grouts (up to 4.0) and «grout to refusal» procedures, the latter expression meaning that grout is pumped into the rock until the maximum pre-determined pressure is reached and no more grout can be injected.
- Use of Bentonite in the grout to reduce separation (also called bleeding) and to lubricate delivery lines. Bentonite is a special kind of clay with favorable properties in this respect.
- Use of stage injection (in terms of depth from surface), low injection pressure and split spacing techniques (new holes drilled in the middle between previous holes). One method of stage injection involves drilling to a certain depth and then injecting the grout for that length. Afterwards, this length gets re-drilled and the hole made longer, followed by a new round of grouting. This process is repeated in steps until full length has been reached. Split spacing as described above is a different way of carrying out staged injection. Holes drilled to full length may also be stage grouted by moving the packer in stages up the hole, or down the hole using a double packer.

The typical overall effect of the above mentioned basic approach was that injection operations were quite time consuming: WPT every 5m; pumping of a lot of water for a given quantity of cement; the need for counter pressure (i.e. grout to refusal) causing unnecessary spread of grout; holding of constant end pressure over a period of time (typically 5 – 10 minutes) to compact the grout and squeeze out surplus water; slow strength development and complicated work procedures. The last point arises from the constant change of w/c-ratio during the pumping to thicken the grout and reduce unnecessary spread. It all amounted to a very long execution time. Due to the very limited maximum grouting pressure being allowed, the efficiency of the individual grouting stages was be limited. This lead to more drilling of holes and further injection stages to reach a required sealing effect or ground tightness. Figure 1.3 (below) demonstrates that a lot of grouting would be carried out at less than 5 bar pressure.



Line	Description
1:	Solid rock
2:	European rule of thumb
3:	Medium solid rock
4:	Weak rock
5:	American rule of thumb
6:	Very unstable rock

Figure 1.3 Relation between rock cover and admissible grouting pressure [1.3]

Summary: The traditional cement injection technique, as described above and for the reasons given, is rather inefficient when considering the time needed and the resources spent in reaching a specified sealing effect. Time efficiency is particularly important when considering work at a tunnel face. The rock cover and limited free surface area would normally allow the use of much higher pressure without the same risk of damage. The tunnel environment therefore requires a different approach.

1.5. Rationale for the increased use of pressure grouting

In the last 20 years, pressure grouting ahead of the face in tunnels (pre-excavation grouting) has become an important technique in modern tunneling. There are a number of reasons for this:

- Limits on permitted ground water drainage into tunnels are now frequently imposed by the authorities for environmental protection reasons and sometimes to avoid settlement above the tunnel. Settlement may cause damage on the surface, e.g. to infrastructure such as buildings, roads, drainage pipes, supply lines, cables and ducts. See Figure 1.4.

Figure 1.4 Northern Puttjern drained by Romeriksporten tunnel (Photo SCANPIX)

- The risk of major water inrush, or of unexpectedly running into extremely poor ground, can be virtually eliminated (by systematic probe drilling ahead of the face, which is an integral part of the pre-grouting technology). It should be noted that if the excavation process hits a large water feature (that was not detected and not pre-grouted), then water ingress has to be sealed in a post-grouting situation. This process is not only time consuming and expensive, but is also far less effective than pre-grouting. In difficult situations it can be close to impossible to successfully solve the problem.
- Poor and unstable ground ahead of the face can be substantially improved and stabilized before exposing it by excavation [1.4]. This improves the face area stable stand-up-time, thus reducing the risk of uncontrolled collapse in areas of poor ground.
- Risk of pollution from tunnels transporting sewage or other hazardous materials can be avoided or limited: once the ground has been treated by pre-injection it becomes less permeable, and such hazardous materials cannot freely egress from the tunnel.
- Sprayed concrete linings are increasingly being used as the final and permanent lining in tunnels. The cost saving potential in construction and time is substantial, this being the main reason for increased interest and use. Such linings are difficult to install with satisfactory quality under wet (flowing water) conditions, and ground water ingress control by pre-grouting can solve the problem.

With modern tunneling drill jumbos, even very hard rock can be penetrated at a rate of 2.5 to 3.0 m/min. Therefore, the cost of probe drilling to protect against sudden catastrophic water inflows is now much lower than it used to be. At the same time, it should be noted that a number of projects have experienced such catastrophic situations, resulting in work being stopped for months. These events therefore become extremely expensive.

With this background it is quite a surprise that the low insurance premium of limited probe drilling is not paid. For a small investment the serious consequences of possible huge water inrush can be eliminated. One should consider that when such conditions are identified ahead of the tunnel face, they can be treated successfully at a fraction of the cost and time spent by just blasting into it. A complete list of examples would be too long, so a limited selection is shown in Table 1.1 (expanded by the author based on Fu et al, 2001).

Table 1.1 Some examples of water inrush at the tunnel face [1.5]

Project Name	**Length (km)**	**Ingress m^3/min**	**GW head (bar)**	**Location**
Pinglin	12.8	10.8	20	Taiwan
Yung-Chuen	4.4	67.8	35	Taiwan
Central (E Portal)	8	18.6		Taiwan
Seikan	53.8	67.8		Japan
Semmering pilot	10	21		Austria
Gotthard Piora pilot	5.5	24	90	Switzerland
Isafjordur	9	150–180	6–12	Iceland
Abou	4.6	180	22	Japan
Lungchien tailrace	0.8	81		Taiwan
NW Himalaya	10	72		India
Oyestol access		5 (single hole)	50	Norway
Kjela		15	23	Norway
Ulla forre		40	20	Norway

1.6. Some comments about post-grouting

Grouting behind the tunnel face (post-grouting) should be used only as a supplement to pre-grouting, to seal off possible spot leakages that may occur. If the ingress requirements are very strict, it may be the case that a section of the tunnel shows an average ingress above the allowed maximum. Post-grouting has turned out to be quite effective when the same area has been pre-injected well, so that only local spots of water ingress can be observed. The normal problem of leakage points shifting from one location to another without really sealing them off is mostly avoided.

It has been repeatedly experienced in a number of projects that post-grouting alone can rarely produce the targeted result, or such results can only be achieved after a prohibitive use of resources. When a certain level of tightness is specified, it cannot be overemphasized that pre-injection is the only reasonable solution, as this process seals open joints in the rock before the water starts to flow, while with post-grouting the water has already started to flow into the tunnel and the joints have to be blocked with pressurized water flowing through them. This can be likened to pumping grout into a fast-flowing creek, hoping to stop it.

One of the problems with post-grouting is grout 'wash-out' and loss of material. A study summing up some Norwegian projects indicates that the time and cost of reaching a specified result by post-grouting will be much higher than by pre-grouting [1.6]. A translation from Norwegian of the two last sentences of page 3 of this reference reads:
«However, it is recommended in cases where large water inrush can be expected and especially at high water head, to carry out probe drilling ahead of the face, and to carry out pre-grouting if large water flow is detected. Based on experience, the cost of stopping water ingress by post-injection is 30 – 60 times higher than that of using pre-injection.»

Other experienced engineers may be using different figures to illustrate the extra cost of using post-grouting exclusively, such as 2 – 10 times more. An accurate figure does not exist, so the important point to note is the general agreement that post-grouting is both extremely expensive and complicated.

When pumping a grout into rock formations, the flow of the grout is governed by the principle of least resistance. The shortest flow path in post-grouting, offering least resistance, very often leads back into the tunnel. To achieve spread of grout into the rock volume, backflow has to be stopped first. Furthermore, if a potential backflow path also carries pressurized flowing water, the injected grout will obviously suffer dilution and wash-out effects. The more water, the higher the pressure and the larger the flow channels are, the more difficult it will be to seal them off.

If the intersection between the flow channel and the borehole is also close to the tunnel wall, this adds to the difficulties. These are the very reasons for the dramatic cost difference presented in reference [1.6]. See also Figure 1.5.

Figure 1.5 Very difficult to seal off by post-injection (photo by courtesy of Oxford Hydrotechnics, UK)

In the Tunnels and Tunneling International issue of June 2005, Mr. Beitnes presents an article titled «Lessons learned from long railway tunnels in Norway» [1.7]. The dramatic background for this article can be quickly summed up as follows: The 14 km long tunnel encountered a zone of less than 2 km length that gave serious ground water ingress control problems. A major part of the grouting work was carried out as post-grouting, causing the overall total tunneling cost to roughly double compared to the bid price, with a considerable increase in construction time.

This author presented a letter to the Editor in Tunnels and Tunneling International of August 2005. The complete letter reads as follows:

Together with Mr. Beitnes, the author of the mentioned article, I was participating in the expert team called in to assist in the design of possible remedial measures after the water ingress problems had caused an intermediate stop in the Romeriksporten project. The referenced article presents a good overview of the project and the problems associated with gaining ground water control. However, I would like to add some comments to the article to enhance the most important lesson learned, as I experienced the situation.

The author states that post-grouting may cost «up to 20 times more» than pre-grouting if used to achieve similar permeability results. An analysis prepared by Olaf Stenstad, presented in his paper at a Post Graduate Course in Fagernes in 1998, states that this factor can actually be as high as 30 to 60 times more expensive. Stenstad based his analysis on material collected whilst working for a specialist grouting contractor, and he came to this conclusion based on experience from several different projects. The motivation to focus on pre-grouting should be increased, understanding the tremendous impact of these figures.

In the article it is correctly stated that «the influence of high groundwater pressure has also been observed, and shows that even when pre-grouting is difficult, **post-grouting may become an even bigger challenge**».

Given that the title of the article is about the lessons learned from Romeriksporten, I would like to add that the most important of all lessons can be spelled out even more clearly: *By **not** executing sufficient pre-grouting to achieve the targeted ingress criteria, the problems caused by difficult ground conditions and high ground water pressure were multiplied several times by doing part of the job as post-grouting. This was in my opinion the overriding reason for the dramatic magnitude of cost and time overrun experienced at the Romeriksporten tunnel.*

How could this happen? There was no relation to the level of geological investigations, as clearly and correctly stated by the author: «In this case, an extensive program of seismic profiles, detailed mapping from core drillings and even permeability tests, would not have substantially changed the tunnel design or procedures, nor prevented the poor grouting and damages resulting from construction». However, when the recommended actions (as lessons learned) are focused around risk analysis and risk assessment, vulnerability studies, and application of methods to predict the degree of difficulty achieving permeability reduction, it is not realistic that any of these measures contribute significantly if the pre-grouting is not planned and carried out appropriately. It can be fully agreed that, as the author states, «considerable uncertainty will remain».

The article states that the original water ingress limits were reasonably correct as shown by the fact that the final approved ingress limits were established at the same level. At the time, difficult ground conditions were encountered, and the volume of grouting works, cost and time consumption increased rapidly. The client, being responsible for the grouting works, the construction budget and meeting the deadline of opening the train service to the new Oslo airport then under construction, ended up taking shortcuts. Pre-grouting work was not continued until ingress results were verified as satisfactory by measured water in-flow from control holes. The blasting of excavation rounds therefore took place prematurely.

The author clearly states the facts: «Despite this huge effort [in pre-grouting], inflow requirements were far from achieved in certain sections». The fact that inflow levels in control holes «seemed fine» cannot be used as proof of successful pre-grouting, when overflow stations in the tunnel invert showed 3 – 4 times the acceptable ingress level. Invert flow measurements are normally a mandatory part of the decision making process, and the total flow along the invert must be used to correct the control borehole ingress criteria if necessary. With a spacing of measuring dams of e.g. 100 m and systematic control and feedback to the grouting program and control hole criteria, it is not possible to excavate 2.2 km of problematic ground without detecting that overall ingress is far above the accepted limits.

It is highly probable that the difficult ground conditions encountered in Romeriksporten would have caused cost and time overruns even if ground water ingress control had been correctly executed by pre-grouting. However, the consequences would probably have been only a fraction of what they turned out to be. It is also easy to understand the pressure on the project management when encountering unforeseen conditions and serious budget and schedule overruns, even though this is not an excuse. The pitfall is always «let us get excavation progress [looks like time saving] and deal with the water later».

In his article, Mr. Beitnes presents a number of good ideas for improvements that can be implemented for pre-grouting in difficult ground conditions and to avoid tunneling negatively influencing the ground water regime and the environment. Many of these ideas are linked to geological investigations, risk analysis and grouting technology, and deserve the attention of the tunneling industry. However, it is of utmost importance to understand that if ground water ingress limits have been defined, these limits can only be satisfied with reasonable cost and time allocation by pre-excavation grouting. Post-grouting has proven not to be an appropriate option on several occasions.

1.7. New material technology allows time saving procedures

The characteristic situation in all modern tunneling is that the rate of tunnel advance is the single most important factor for the overall tunneling cost. This fact is closely linked to the very high investment in tunneling equipment, causing high equipment capital cost. Added to this is the fact that there is only limited working space at the tunnel face, only allowing one work operation to take place at a time.

The face advance rate is decided by the number of hours available for excavation works (other factors kept constant). Time spent for pre-injection will normally have to be deducted from this available excavation time. One hour of face time may easily have a value of $ 2000 US and it is evident that the efficient conduct of all activities at the tunnel face will have top priority. From this it can be seen that injection in a tunneling environment is fundamentally different from injection for dam foundations and ground treatment from the surface. This is the main reason and driving force behind the different technical development in tunnel injection as compared to the mentioned surface based applications.

Due to the need to save time (and therefore cost), technical specifications for routine tunnel grouting cannot be loaded with tests and investigative techniques. Whether extensive water pressure testing in stages and in all holes is required, whether core drilling is made part of the routine drilling from the face, whether joint orientation and crack openings have to be checked by camera etc., is all linked to a complicated system of decision-making during the execution of grouting. The sum may easily be termed “overkill”. Such research related activities cannot be made part of the routine grouting works if cost and efficiency have any priority. The sad part of this is that such over-zealous procedures will probably not improve the end result at all.

The last 20 years has led to the development of a number of new cement based products for injection. Typically, these cements are ground much finer and may offer more suitable setting and hardening characteristics. In most cases, these cements are combined with admixtures or additives to provide entirely new cement grout properties and substantially improved penetration into cracks. When combined with working procedures that are adapted to the new material properties, the efficiency increase is substantial. Another important element in these new procedures is the ability to drill long holes at high penetration rates.

Even though these new cement products are more expensive than standard Portland cements available locally, they are still very competitive when total cost is considered. They also cost less than traditional chemical grouts.

Cement based grouts remain the material of first choice for pressure grouting in tunneling. This is due to the low volume cost, the availability, the well documented properties and the experience and environmental acceptability. The wide range of available chemical grouts offers a useful supplement to cement grouts, especially when tightness requirements are strict. Chemical grouts can penetrate and seal cracks that cementitious grouts will not enter.

The last new development in material technology for grouting is the colloidal silica, or mineral grout. This suspension of nanometric particles behaves like a true liquid and has opened a wide new field of opportunities in the grouting of soil and rock. More on this topic will follow in chapter 12.

2. GROUTING INTO ROCK FORMATIONS

2.1. Particular features of rock in comparison with soil

Almost all rock formations are fundamentally different to most soil deposits when considering flow of ground water and any pumped grouting material. What can be achieved and how to execute rock injection is therefore also very different to any grouting operation in soil.

Soils possess a wide variation of particle sizes, layering, compaction, porosity, permeability and a number of other parameters. However, at a basic level soils consist of particles and the permeability is directly linked to the pores (spaces or voids) between the individual particles.

Between discontinuities, most rock materials are practically impermeable for water and grouts. Leakage and conductivity is therefore linked exclusively to discontinuities within the rock mass. It is vital to understand and accept this important difference between soil and rock to be able to correctly evaluate all aspects of pressure grouting in rock tunneling, and to understand why the approach has to be different to soil injection techniques.

When comparing rock and soil, the similarities and differences are primarily governed by how scale is being treated. It is important to understand and take account of the effects of scale to reach correct solutions and answers. If the conditions within the whole mountain are considered, the average «permeability» of the rock mass can be measured and evaluated using the same methods as are normally used for soils (similarity). The reason for this is that the overall rock mass fragmentation creates relatively small block sizes (similar to particles in the soil case) when looking at the whole mountain volume. Therefore, the whole rock mass can be treated as having an average permeability when viewed at this scale.

In comparison, when considering the rock volume for the first few meters around a tunnel and along a few meters of its length, single joints and channels will dominate the pattern of water conductivity and grout take.

In a randomly chosen limited rock volume, the joints and channels can show water conductivity many orders of magnitude larger (hundreds or thousands of times larger) to the «mountain» average permeability (difference). Using the term permeability when describing rocks, the same way as for soils, can therefore be highly misleading.

In a perfectly homogeneous sand volume of a given permeability one could, as an example, calculate 300 l/min of water ingress into a 100 m tunnel length. If we alternatively assume that the sand is impermeable but it has a distinct local channel leading into this same tunnel section, the channel could also feed 300 l/min into the tunnel. The channel scenario could be an illustration of a hard rock tunnel with an open water conducting channel. The average «permeability» over the 100 m tunnel would be the same in these two imaginary cases. However, the two situations are totally different in practical terms if looking for a solution to seal off the water ingress.

The permeability term is also being used to estimate and illustrate general ground water flow conditions on an overview level in hard rock (large scale average), and this is an acceptable approximation. When changing to a more detailed level of observation in a rock situation, the term permeability is not applicable any more. Practical decisions made based on an assumed «permeability» will usually turn out to be totally wrong.

For injection in **soils** the following indications have been given by Karol [2.1]:

$k = 10^{-6}$ or less	not groutable
$k = 10^{-5}$ to 10^{-6}	groutable with difficulty in grouts under 5 cP viscosity and not groutable for higher viscosities
$k = 10^{-3}$ to 10^{-5}	groutable by low-viscosity grouts but with difficulty when viscosity is more than 10 cP
$k = 10^{-1}$ to 10^{-3}	groutable with all commonly used chemical grouts
$k = 10^{-1}$ or more	groutable by suspended solids grouts

(It should be mentioned that at the time of publishing this reference, nanometric colloidal silica was not available).

Based on the previously mentioned differences between soil and rock, the above guidelines are not applicable in most rock materials. With WPT results in boreholes as a basis for calculation of permeability in rock, even section lengths as short as one meter could easily indicate permeability between one and three orders of magnitude too low (10 to 1000 times too low). The measuring lengths are mostly longer and the calculated permeability could be even further from the norm. In addition, the fact that rock injection in tunneling allows the use of much higher injection pressure (often 10 times more) will change the practical limits of what is and is not groutable.

In a rock mass it is evident that the characteristics of jointing will be of major importance for any grouting program. The variation of joint properties and water conductivity in different types of rock is extreme, and a discussion of this subject in any detail is outside of the scope of this book.

However, some examples can be given to illustrate the importance of the subject and to draw attention to some effects of typical conditions found in rock. Certainly the most extreme water conductivity situation in rock is linked to limestones with karst features. These are solution channels in limestone formations that can create huge caverns and literally allow the river to go underground. Even if the channel has a typical diameter of just a meter or two, the water flow conditions into a tunnel intersecting it would be catastrophic.

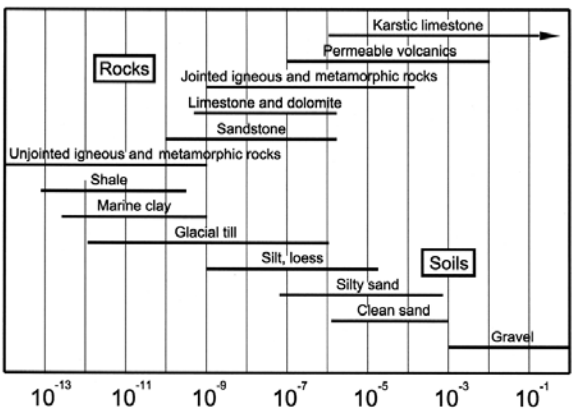


Figure 2.1 Average permeability of soil and rock

Hard rock materials like gneisses, granites and quartzites, will often show unweathered jointing patterns at a depth that may result in a substantial total leakage potential. Such jointing can be quite easy to inject and seal. Local fault areas, especially major shear zones in the same kind of bedrock, may contain a lot of fine material and clay gouge. Such zones or larger rock volumes subjected to tectonic movements will often show small or no leakage due to all the fine material in the joints. There are also cases where such movements have produced extensive jointing and weathering with lots of fines, but still with a generally high ground water conductivity. Grouting under such conditions may become very difficult and uncontrolled ground water flow may cause flushing out of fines and increasing water ingress with time.

Weaker bedrocks like shales, limestones, mudstones, sandstones and some metamorphic rocks are frequently layered and jointed to a considerable degree. A high number of water-bearing small cracks may in total produce substantial leakage. A complication for a successful injection program in such rock conditions is often the wide variety of joint filling materials available. Such joint fillings tend to inhibit grout penetration and distribution and the fill materials are sometimes squeezed around by the grout being injected. See Figure 2.1.

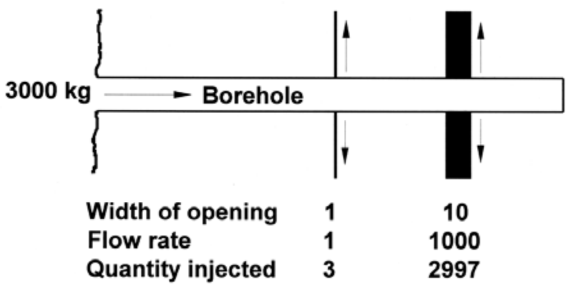


Figure 2.2 Effect of conductivity contrast on grout flow into open joints

In most rock masses the main problem for pressure grouting is the non-uniform conditions caused by localized geological features. In a borehole with a length of some meters there will, in most cases, be a mixture of joints, cracks and channels, and more or less watertight sections in-between. Any fluid pumped into such a borehole will inevitably follow the path of least resistance. The effect of this is that a given volume of grouting material may follow a very conductive opening at fairly low pressure, to a distance much greater than expected and beyond what makes any practical sense. At the same time there will be very limited penetration into other openings (due to the low pressure and material «lost» into the main channel). This problem can and very often does lead to unsatisfactory grouting results and increased cost, due to an increased number of grouting stages and too high material consumption to achieve the required result. See Figure 2.2 above.

In a rock type with only one dominating joint set, where one would expect water ingress and grout penetration to generally flow along these joint planes, this will only partly occur. Observation of the nature of water ingress in TBM excavated tunnels (where additional blasting cracks are not obscuring the natural conditions), clearly demonstrates that water bearing channels within joint planes are the typical situation. This is well demonstrated by leakages appearing as concentrated point «jets» from somewhere along the joint intersection with the tunnel periphery.

Experience from post-grouting in tunnels further supports the idea of channel conductivity as the normal mechanism of water transmission in jointed hard rock. When a water flow clearly originates from an identified joint plane that can be observed crossing the tunnel periphery, drilling can be performed to cut through the joint plane at a suitable depth and angle, with the purpose of getting direct contact to the water flow. Often a number of holes need to be drilled across the joint plane to actually find the water. The reason is obvious – most of the joint plane is dry and the water flows through a limited size channel within the plane. When drilling for water flow contact, it is obviously much more difficult to hit a «pipe» than a plane.

An example can be given from the Norwegian hydropower project Kjela (1977). At the tunneling length of 1800 m from the access tunnel Tyrvelid, direction Bordalsvann, the tunnel hit a water inrush of 15000 l/min at 23 bar pressure. As could be clearly seen in the tunnel, more than 90% of this inrush came from one concentrated channel located within a shear zone.

2.2. Handling of rock conductivity contrast

Because of the need for time efficiency when carrying out pre-excavation grouting, the holes are normally quite long (10 to 30 m) and they are grouted from one single packer placement close to the face (1 to 3 m). In such a length of borehole there will be conductivity contrast along the hole, and sometimes this contrast may be extreme. With a large conductivity contrast and grout flow in the direction of least resistance it is necessary to take steps to reduce the negative effects of this quite normal situation. See Figure 2.3. Due to time and cost reasons, this problem cannot be solved by multiple packer placements to grout only short sections at a time.

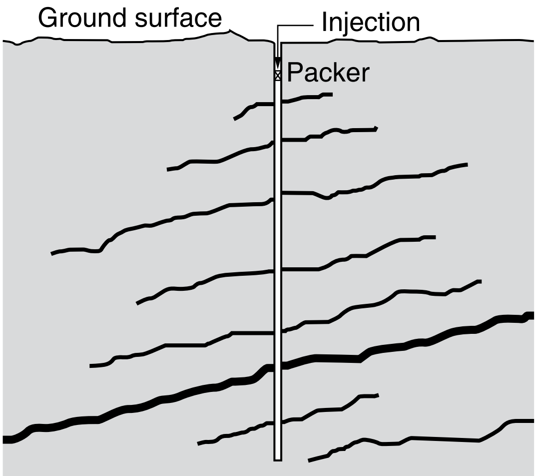


Figure 2.3 Large conductivity contrast

The problem is that chemical grouts will flow into the large openings at low pressure and nothing or very little will enter and seal smaller openings. Cement grouts will have the same tendency and grout to refusal (to increase the pressure), giving excess material consumption, but the fine cracks become clogged when the pressure finally increases. Stable cement grouts and suitable procedures can counteract the problem to a large extent and thus increase efficiency. The best way to illustrate how to deal with conductivity contrast is demonstrated in Figure 2.3. This situation can be treated with traditional grout to refusal technique and alternatively with stable cement grout and dual stop criteria.

2.2.1. Description of typical grout to refusal procedure

Start of grouting with a w/c-ratio of 3.0, high grout flow at very low pressure and assuming that 90% of the flow goes into the largest channel. Standard procedure would be to reduce the w/c-ratio in steps when the pressure is not increasing. One could assume that after 3.5 hours injection time, circa 4000 kg of cement has been pumped, reaching the maximum allowed pressure (for the specific local conditions). The following situation is reached:

- Cement has traveled in the largest channel to a maximum distance of 350 m from the borehole (which is far beyond the useful spread).
- The grout pressure increases gradually, especially during the last part of the injection time, when using the thickest grout.
- Grout permeation into medium and small cracks is only in mm-scale. This is caused by a long period of time under low pressure and clogging of the cracks by filter cake development. Furthermore, when the pressure finally increases the grout used has a low w/c-ratio and higher viscosity and will therefore not permeate small openings.
- Some of the injected grout has separated (bleeding), leaving residual openings and conductivity.

2.2.2. Stable grout of micro cement using dual stop criteria

The whole injection can be executed with a fixed w/c-ratio of about 1.0 and a low viscosity of 32 seconds Marsh cone flow time, using a thixotropic grout. In this case, 90% will also flow into the largest channel at very low pressure. After one hour of injection time the stop criterion of 1500 kg of cement has been reached (pressure still low) and the grouting stops. (In a real case under such extreme conditions, micro cement would be injected at w/c-ratio 1 for a very limited time, circa 250 kg before changing to 0.8 and may be 0.6). The established situation may be assumed to be as follows:

- Micro cement has traveled on the largest channel to a maximum distance of 125 m from the borehole (which is also beyond the useful spread). This shorter distance is primarily caused by less cement being pumped (stop criterion on maximum quantity).

- Some penetration has been achieved in medium size openings due to the grout stability, low viscosity and small particle size.

Assuming that the hole length is 12 m, the next step is to drill a new neighboring hole with the same length. This takes about 5 to 10 minutes with modern drilling equipment. Injection can now take place in the same area where the large channel is blocked by first stage injection, and penetration into medium and small cracks will occur at a higher injection pressure. It can be assumed that it takes 30 minutes to inject another 500 kg of micro cement to reach the allowed maximum pressure.

2.2.3. Comparison of the two procedures

	Traditional OPC grouting	**Stable micro cement grouting**
Time spent	3.5 hours	1 hour 40 minutes
Materials consumed	4 000 kg OPC	2 000 kg micro cement
Injected	1 stage, one crack	2 stage large and small cracks
Result	Ineffective	Mainly effective

The micro cement alternative using half the material and less than half the execution time has achieved the following result improvements compared to the OPC procedure:

- As two grouting stages have been executed, the achieved rock tightness in the first meters around the hole is much better. Other reasons for better tightness are the fact that the grout viscosity was very low, the grout was stable (no recreation of channels due to bleeding) and the maximum cement particle size would typically be 1/4 of the OPC.
- The grout durability and strength is substantially better because of the lower w/c-ratio and no use of Bentonite in the mix.

It would also be an option to execute two grouting stages using OPC and then the result could of course be improved. However, this would take additional grout and additional time, and experience shows that the result would still be poorer. The cost of extra cement and even more importantly, the extra time will normally cause substantially higher overall cost for a poorer result using OPC and grout to refusal technique.

2.3. «Design» of grouting in rock tunnels

Design of grouting in rock tunnels essentially covers the development and specification of drilling patterns, the grout materials to be used and the methods and procedures to be applied during execution. These are the variables that can be controlled by engineers, geologists or specialists and which are varied according to local conditions in the tunnel, with the purpose of achieving a specific result.

The outcome cannot be accurately predicted because of the nature of the technique and the lack of details about ground conditions. Nobody can directly observe what happens in the ground during injection, other than the indirect signs (grout process monitoring) and effects on water ingress as well as by inspection after excavating the grouted rock volume.

Even the evaluation of carefully controlled full scale tests can be difficult. The uncertainty in relation to changes in ground conditions from one test location to the next cannot be accurately quantified. However, most of the principles of pre-grouting have been developed through and supported by the results of several thousand tons of grout injection material in tunneling, and the understanding of the principles is not as much guesswork as it is sometimes claimed to be.

The word «design» probably needs to be commented upon to clarify what it means in the context of tunnel grouting. The need for such a clarification arises from the difference to the normal understanding of the term when used in e.g. structural design.

Design of a bridge or a high-rise building will include the necessary drawings, material specifications and structural calculations to define the dimensions, the geometry, the load-bearing capacity, the foundations and the general layout of the object to be built. The whole analysis has to be based on the given physical surroundings, the owner's requirements regarding service loads, service life expectancy and other features or limitations that are applicable.
In the case of a tunnel grouting operation many will expect the above basic principles to be applicable as far as the «design» process is concerned. The reality is that it is not possible to design the work with precision in advance of it being carried out, so it is nothing like the

«design» process referred to in the previous paragraph. The design of tunnel grouting operations is based upon the best estimates of the average «permeability» of the rock through which the tunnel is to be driven. The design will usually include calculations of the likely water ingress with and without grouting, drawings showing matters such as depth, angle, and pattern of intended drilling, execution procedures covering all aspects of the operation and the material specification, aiming to satisfy the required water tightness of the tunnel. Drawings showing what the finished job will look like or giving accurate dimensions and quantities for the result are not possible.

The execution will vary from location to location based on information that is obtained from the progressing work (observations of water ingress from boreholes, pumping pressure and quantity per hole during grouting, water ingress from verification holes etc.). A well planned grouting operation will have the necessary built-in flexibility to cover local variations in hydro-geological conditions.

The pre-investigations for rock tunnel projects can never give sufficient details about the rock material and the hydrogeological situation for the full length of the tunnel, so as to allow a «bridge design» approach. Furthermore, the calculation methods available are not refined enough to accurately analyze the link between the required result and the necessary steps to produce it. To further compound the problem, even if an accurate mathematical model would be available, there is no chance that all the material parameters could be measured, accurately quantified and input to such a model.

The basic design for the grouting operation as referred to above has to be applied in practice on an empirical, iterative, observational design-feedback basis (monitoring of actual results) as described below:

- Once the «water tightness» requirements are defined, the project data and all available information about rock conditions and hydrogeology can be analyzed and compared with those requirements. This often includes indicative calculations of potential ground water ingress under different typical situations. Based on empirical data (previous pre-injection tunnel project experience) a complete pre-grouting method statement can be compiled. However, irrespective of how elaborate this method statement (or «design») is and whatever tools and calculations are employed to produce it, it will not be more than a prognosis for the future work. This prognosis will express how to execute the pre-grouting (under the expected range of ground conditions), and what sequence of steps to take in order to meet the required tightness of the excavated tunnel.

- During excavation the resulting tightness in terms of water ingress achieved can be measured quite accurately. This means that it is possible to move to a quantitative comparison between targeted water ingress and the actual result and accurately pinpoint if the situation is satisfactory or not. If the results are satisfactory, the work will continue without changes, and only a continued verification of results by ingress measurement will be necessary.
- If the measured water ingress rate is too high, this information will be used to decide on what steps to take to improve this section and how to modify the «design» to ensure satisfactory results in similar sections not yet excavated. The improvement works may have to be executed in stages, until satisfactory ingress values can be measured, and will normally consist of local post-grouting.

2.4. Fluid transport in rock

The permeability of a material expresses how readily a liquid or gas can be transported through it. Darcy's Law is based on laminar flow, an incompressible liquid with a given viscosity, and is valid for homogeneous materials [2.2]:

$$v = k\, i$$

Where
v = flow velocity
k = coefficient of permeability
i = hydraulic gradient

The requirement of homogenous material is never satisfied for jointed rock materials, and can be approximated only when the volume being considered is large enough. Normally, the term "joint permeability" or even better "conductivity" should be used.

The coefficient of permeability can be measured in the laboratory, using the above formula by Darcy:

$$q = k\,A\,i$$

where q = liquid flow rate (m^3/s)
k = coefficient of permeability (m/s)
A = area of sample across flow path (m^2)
i = hydraulic gradient

The absolute permeability of materials for liquids of varying viscosity can be found according to the following formula:

$$K = k\,(\mu/\gamma) = k\,(\nu/g)$$

where K = absolute permeability (m^2)
k = coefficient of permeability (m/s)
μ = dynamic viscosity (mPa s or cP)
ν = kinematic viscosity (m^2/s)
$g = 9.81\ m/s^2$
γ = volume weight of the liquid (N/m^3)

For the testing of rock mass conductivity through boreholes, the unit Lugeon is most frequently used. Lugeon (L) is defined as the volume of water in liters that can be injected per minute and meter of borehole at a net over-pressure of 10 bar (see Figure 1.2)

The Lugeon value needs interpretation and cannot be considered in isolation. If measurement has taken place over a borehole length of 10 m for example, the total water loss will be averaged over the measuring length. In principle, there is always the chance that all the water has escaped through a single channel location. This means that if this borehole had been measured in 0.5 m increments, nineteen of these would have had an L-value of zero, while one would be 20 times the average value for the full 10 m length.

To avoid any extreme differences between Lugeon values resulting from a single measurement over a long borehole (10 to 30 m) and a more realistic value measured over shorter segments (such as 1.0 m), technical specifications sometimes require that the Lugeon value calculation length is set to 5.0 m for all borehole measuring lengths longer than 5.0 m.

The following table illustrates the different units discussed above:

Table 2.1 Comparison of permeability units
The factor should be on the top

Materials/Units	Lugeon	k (m/s)	K (m²)
Fine sand	100	10^{-5}	10^{-12}
Jointed granite	0.1	10^{-8}	10^{-15}

2.5. Practical basis for injection works in tunneling

Pre-injection in tunneling may have various purposes and may be carried out under widely variable geological and hydrogeological conditions. All these factors will strongly influence how to execute pre-injection in a given case. However, there are a few basic, practical facts when at a tunnel face that must be part of any pre-injection planning and execution.

At a tunnel face, there is often limited working space and the logistics may be an added problem. Working operations at the face are mostly sequential, and very little can be executed in parallel. To keep the cycle time short and the rate of tunnel face advance high, it is extremely important that all work sequences are as rapid as possible, with as little disturbance and variation as possible and with a smooth change from one operation to the next. This is obviously decisive for the cost of the tunnel, as time related expenses are running whether there is face advance or not.

One very important aspect of tunnel face injection must be emphasized. In general, injection into rock materials is not an easily pre-planned activity. Pre-investigations may have yielded a lot of general information, but very little on a detailed level. On the other hand, a lot of specific and detailed information is generated during the drilling of holes and during execution of the injection itself. The temptation on the part of planners and designers to create very elaborate working procedures, a lot of tests, voluminous record keeping and tight supervision is therefore very strong. If such a tendency is not evaluated against cost/benefit (the time it takes and the value of the information), this can generate very complicated and time consuming decision procedures. A lot of detailed information must be processed with clear lines of authority, and decisions must be made regarding the influence on further and future work operations. It is very easy to end up in a situation where good technical intentions turn out to be negative to the purpose of the exercise.

Elaborate WPT procedures with the purpose of choosing the type of grout are frequently relied upon far beyond the technical merit of the procedure. Plotting of experience data to check on the possible correlation between grout take and the originally measured Lugeon value will be very disappointing. One example of such data-plotting is shown in Figure 2.4. All such efforts that the author has come across are similar to what is shown in this figure.

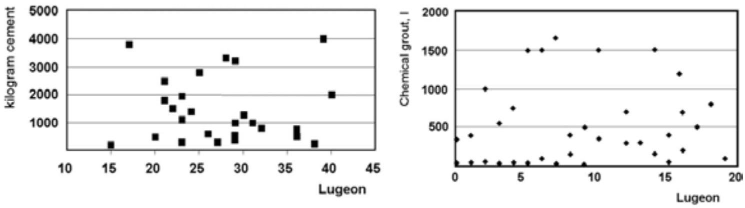


Figure 2.4 Correlation between measured L-value and grout consumption [2.3]

It is possible to map the variation in borehole conductivity by executing WPT in short sections, and in theory, this may be used to adapt the grout type or properties to this variation. One should also expect that grout take needs to be adjusted differently to sections with high conductivity, as compared to low conductivity. The reality of real life tunnel grouting is that all this takes a lot of time (high cost) and the benefits are highly questionable, partly due to the reality of Figure 2.4.

Another basic aspect of pre-injection must be kept as part of planning and operation. Regardless of the reason for the pre-injection, as long as it has to do with ground water control it will be very difficult (or unrealistic) to hit exactly the targeted residual ground water ingress everywhere and with accuracy (see chapter 2.3 «Design» of grouting in rock tunnels). This is the case whether the target is very strict, such as 2 l/min and 100 m tunnel, or 10 times this level. There is no feasible way of substantially improving this lack of accuracy and there are therefore clear limitations to the level of refinement and sophistication that would make it reasonable and productive to add to the injection procedures.

This may seem very negative and could be understood as a complete lack of control of the injection process. One might be tempted to state that it makes no sense to execute pre-excavation grouting when such uncertainty exists as to what result will be achieved. This is fortunately not the case, due to two main factors:

- Water ingress measurements in the already excavated tunnel parts will tell where the criteria are not met, to what extent the results are inexact, under what conditions, and resulting from which resource allocation already used in probe drilling and pre-injection. The same goes for the tunnel sections with satisfactory results. This information and its evaluation can be continuously fed back to the at-face execution for necessary correction of procedures. Experience shows that the targeted results will then be more closely reached and with a more optimal use of resources.
- In those areas where the criteria are not met, post-injection can be undertaken, normally starting with the highest yield leakage points. This technique is very efficient when pre-injection has already been carried out, otherwise leakage would normally just be moved around. Because of the actions described in the first point, the need for post-injection in subsequent parts of the tunnel is quickly reduced and the final result will meet the specified requirements.

The main conclusion to draw from this information: The targeted final result will be met.

As it is generally much more efficient to execute pre-injection, it is also better to start a little on the conservative side with the works procedures and later to relax the approach as appropriate experience is gained. When requirements are tight and the potential consequences of not meeting criteria are serious, it is often best to simply decide on pre-injection as a routine systematic activity using double cover approach (see Figure 2.5). The rationale is that if probe drilling in most cases will lead to pre-injection, then this separate activity and decision making can be saved, thus simplifying the procedures and increasing efficiency.

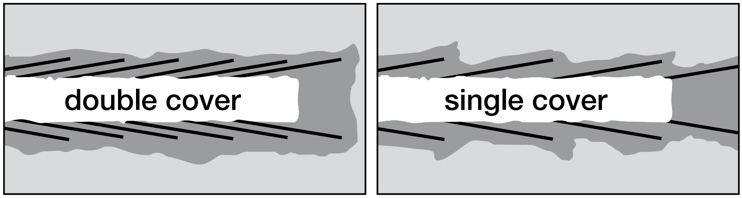


Figure 2.5 Double and single cover grouting pattern

In less strict situations e.g. with a maximum allowed final water ingress of 30 l/min, a 100 m tunnel and no consequences in the surroundings of the tunnel, a limited overlap of typically 5 m per 20 m probing length (25%) can be used. In this case probe drilling will normally be used to provide the basis for decisions on where to actually execute pre-injection. Sections of the tunnel with relatively small water yield from probe holes will then be passed through using probing alone. Where injection is needed, the single cover approach should be used (see Figure 2.6).

2.6. Grout quantity prognosis

Almost all pre-grouting in hard rock tunneling is based on the use of cement (Ordinary Portland Cement, OPC or micro cement).

In special cases, such as in ground conditions with clay and other fine materials on the joint planes and/or when the required tightness cannot be reached with cement only, chemical grouts may be necessary as a supplement. There is no experience basis available for the use of predominantly chemical grout to illustrate typical consumption. However, in the case of cement injection, such experience data is available.

In the case of cement only grouting, the required quantity will depend on a large number of factors, and any estimate made in advance will be inaccurate. The main influence factor is the rock condition (properties of the jointing), where a limited number of large open channels will tend to require more cement than cm-scale joint spacing that produce the same total water ingress. Other important factors are the required tightness, static head of ground water, tunnel cross section and even the type of cement and injection methodology employed.

From sub-sea tunneling with systematic probe drilling and partly with systematic pre-grouting, there are average consumption values from quite variable Scandinavian conditions between less than 20 kg/m tunnel to more than 250 kg/m tunnel. As an extreme case, the Bjoroy sub-sea road tunnel stands out with a section of about 500 m tunnel length consuming 2000 kg/m. Target water ingress level was 30 l/min and a 100 m tunnel.

When evaluating empirical data covering such a wide range it can be useful to view the data on a probability basis. Three different figures can be used to illustrate the experience data available from Norwegian sub-sea road tunneling:

1. Minimum average consumption, with 5%probability that the average will be less than this figure.
2. The probable average consumption.
3. Maximum average consumption, with 5% probability that the average will be more than this figure.

The minimum can be expressed as 15 kg/m tunnel, the most probable value is 50 kg/m tunnel and the maximum average is 500 kg/m tunnel. These values are roughly representative of predominantly hard rock types (but not only granitic rock materials) and the tunnel length would have to be more than 1000 m to yield a reasonable average. Such figures can obviously only be taken as an illustration of what has been experienced before and they cannot be transferred directly and accurately to new projects in other ground conditions.

It must be mentioned for clarity that the above figures are averages for the whole tunnel length, including tunnel sections that need no grouting at all.

3. FUNCTIONAL REQUIREMENTS

3.1. Influence of tunneling on the surroundings

Any tunnel excavated will influence the immediate surroundings to some extent. Depending on the location of the tunnel and its design and purpose, ground conditions, hydrogelogical conditions etc., such influence could cause problems.

The main issues that need evaluation can be listed as follows:

- Purpose of the tunnel and the requirements of lining design (drained or water tight). Most linings are drained, even if there is a horse-shoe shaped umbrella installed to prevent water from dripping on the road or on installations in underground facilities (see Figure 3.1). To produce watertight tunnel linings is very complicated and costly, especially if the ground water head is high. High pressure in this context would be anything more than 5 to 10 bar.

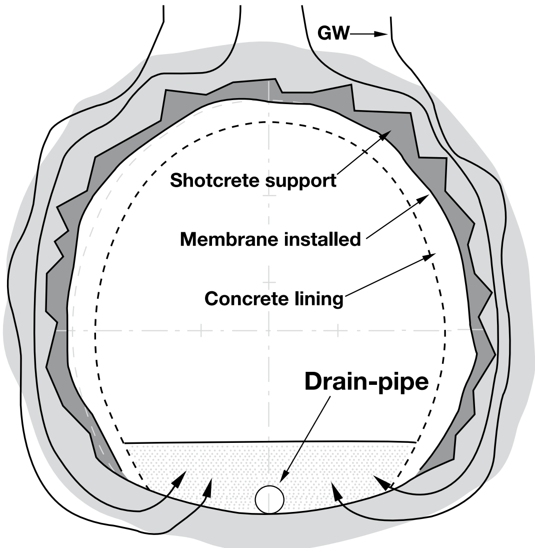


Figure 3.1 Typical waterproofing, drained solution

- Location of the tunnel, especially in relation to other infrastructure, other excavations, lakes, rivers and general ground water level. Most tunnels are below the local ground water level.
- Amount of rock and soil cover, type of tunneling ground and water conductivity of the ground.
- Possible consequences of in and out leakage on the economy, the environment, safety and health. Out-leakage can be as much of a problem as the other way around. Hydropower pressure conduits will lose water and electricity production and sewage may cause pollution.
- Requirements and limitations for the construction phase as well as for the permanent use of the tunnel. These may be quite different.

The possible consequences of tunnel excavation on the surroundings may be listed as follows:

- Ground water ingress may lower the pore pressure in soil deposits above the tunnel and this could cause ground settlement. This is typically a problem where clay deposits lose their pore pressure. With buildings and other structures founded on clay, severe damage may arise. Such problems may already occur at ingress levels of 1 to 5 l/min and 100 m tunnel (see Figure 3.2).

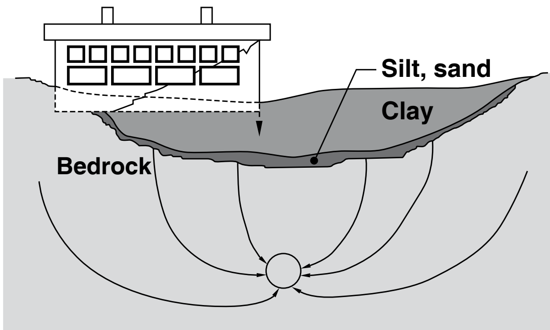


Figure 3.2 Particularly settlement sensitive situation

- Lowering of the general ground water level can have a number of different effects. Ingress of oxygen to wood foundations will cause rotting. Some rocks such as alum shale may swell due to creation of gypsum, causing damage to foundations and other structures. Earth pressure on sewage lines, cable ducts etc., will increase.

- Ground water resources like springs and wells may be influenced or lost, vegetation may dry out and farming activities may be damaged. The extent of ingress that can be accepted depends very much on climatic conditions and the relation between surface run-off and remaining water quantity going into the ground.
- Out-leakage consequences will very much depend on what liquid and the type of components in the liquid that is leaking out and under which hydrostatic head. Water may cause splitting, jacking or washing out effects at high head and water influx at unwanted locations also at the lower head. Contaminated water such as sewage, hydrocarbon liquids, poisonous liquids, gases etc., will in most cases cause severe environmental problems in the surroundings.

3.2. Conditions inside the tunnel

Inside the tunnel, water ingress will cause a variety of problems, but the consequences are different during the construction phase as compared to when the tunnel is in operation:

- In the tunnel construction phase, the problems are primarily of a practical nature. When excavating on a decline, water runs to the face and has to be pumped out. The acceptable quantities are lower for a TBM excavation (where less than 500 l/min at the face will cause serious difficulties) than for drill and blast (D&B) (where 2000 to 2500 l/min may be handled reasonably well) and will also depend on a number of other factors. Tunnels actually being driven on an incline, but with access through a shaft or a declining ramp, will require constant pumping. Naturally, pumping of water may become an important cost factor at high volumes or high pumping head.
- Water ingress can behave in a number of different ways. Concentrated high volume and high pressure inrush may cause flooding and severe problems and time loss (refer to the example mentioned in chapter 2, Kjela). Distributed water ingress and generally wet conditions will also cause problems, such as poor conditions for sprayed concrete application, concrete works, construction road works, construction phase dewatering and drainage, unstable rail tracks etc. Water may have a high or low temperature, causing a very poor working environ-

ment, and it may also contain salt. Salt water produces corrosion and problems with all electric equipment underground.

- Depending on rock type and quality, water can create instability, rock decomposition, rock swelling and washing out.
- In the permanent use of the tunnel, wet conditions will produce similar problems as mentioned above. Typically, during the operation phase tunnels will have technical installations of a different kind, such as the permanent ventilation system, electric supply and operation systems in metro tunnels. Humid conditions will cause corrosion, electric failures and other problems over time. The maintenance and repair cost may become high and the disturbance of operations may be even worse.
- In cold climates and ventilated tunnels, water ingress can cause ice build-up. In most cases this cannot be permitted and has to be taken care of if it occurs. In a traffic tunnel, even local minor drips (less than 1.0 l/min per 100 m tunnel) of minor or no concern above freezing point, can turn into serious problems when the frost volume is high enough.

3.3. Calculation of water ingress to tunnels

Parameter list:

q	ground water ingress flow rate	m^3/s per meter of tunnel
h_s	thickness of soil above the tunnel	m
h_w	height of water above the tunnel	m
h_r	thickness of rock above the tunnel	m
r	tunnel radius	m
k	coefficient of permeability	m/s

Figure 3.3 shows an example situation with the parameters necessary for the calculation of ground water flow rate into the tunnel, including the formula to be used.

The physical significance of the parameters will depend on the actual situation. It is important to note that hr and k represent the thickness of ground where the main part of the potential reduction takes place (energy dissipation or pressure loss).

$$q = \frac{2\pi k\,(h_w + h_r)}{0.6 - \ln\left(\frac{r}{h_r + r}\right)}$$

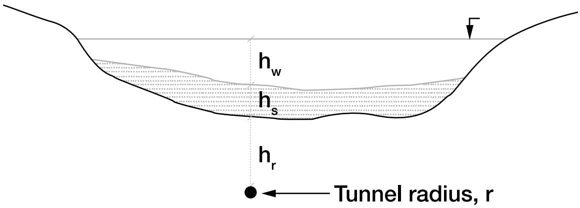


Figure 3.3 Ground water ingress formula

When the soil permeability is much higher than in the rock, then h_w in the above formula must be replaced by ($h_w + h_s$). On the other hand, if the soil is at least as tight as the rock, then h_r must be replaced by ($h_r + h_s$).

When injection has been carried out around the tunnel and the injection zone is substantially less permeable than the surrounding rock mass, then the h_r has to be expressed as the sum of the tunnel radius r and the thickness of the injected zone, while h_w will be replaced by the sum of $h_w + h_s$ + thickness of not injected rock mass (see [3.1]).

A typical situation for an urban tunnel at shallow depth:

In critical bedrock low points filled with sand and marine clay and with buildings on top, a set of assumed example dimensions are shown in Figure 3.4.

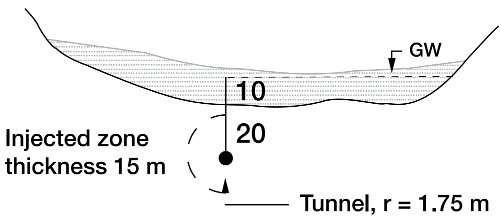


Figure 3.4 Shallow tunnel with soil and rock cover

If the average rock mass permeability in Figure 3.4 is $k = 10^{-7}$ m/s, this would give a water ingress rate of 24.2 l/min per 100 m tunnel (using the above formula).

A typical tightness requirement for such a tunnel to avoid settlement and surface damage could be 5 l / min and a 100 m tunnel, which corresponds to $q = 8.33 \times 10^{-7}$ m^3/s and m. With the dimensions shown in Figure 3.4 and an assumed injection zone thickness of 15 m, the required permeability of the injected rock volume would have to be:

$$h_f = 1.75 + 15.00 = 16.75 \text{ m}$$
$$h_w = 10 + 5 = 15 \text{ m}$$

And entered into the formula it gives:

$$\underline{k = 1.23 \times 10^{-8} \text{ m/s}}$$

To achieve a reduction of the water ingress rate to about 1/5th, the 15 m grouted zone permeability must be reduced to about 1/8th.

A typical deep situated tunnel:

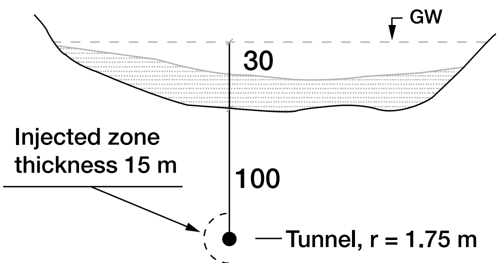


Figure 3.5 Deep situated sub-sea tunnel with soil and rock cover

The assumed dimensions are shown in Figure 3.5. If we assume that the injected zone has the same thickness of 15m as in the shallow case and that the resulting permeability after injection is also the same, then the increased hydrostatic head with the shown geometry would produce a ground water ingress of:

$$h_f = 1.75 + 15.00 = 16.75 \text{ m}$$
$$h_w = 30 + 85 = 115 \text{ m}$$

And entered into the formula it gives:

$$q \quad = 3.44 \times 10^{-6}\ m^3/s \text{ and } m = \underline{20.6\ l/min \text{ and } 100\ m \text{ tunnel}}$$

3.4. Special cases

Water transfer tunnels in hydropower projects are often constructed as drained structures. Even pressure tunnels and pressure shafts may be designed as drained (unlined) conveyors when the rock conditions and the rock stresses allow such a solution. Naturally, there will be local areas where injection has to be carried out to limit the loss of pressurized water and electricity production.

In pressurized unlined water conduits (where the rock has to sustain the water pressure) there is one pre-requisite to be aware of. The minimum principal rock stress must be higher than the water pressure in all locations, otherwise the water will find its way out through cracks and joints in the rock mass and hydraulic fracturing is very likely to occur. In such a situation, normally leading to a substantial loss of water, the option of grouting as a method of repair is ruled out. Grouting may temporarily help reduce the flow, but the risk of another fracture somewhere along the tunnel (or shaft) is quite high. The expression «hydraulic fracturing» is used here to describe a failure that could be either hydraulic lifting or jacking on existing discontinuities, or an actual hydraulic fracturing through solid rock. Jacking is the most likely scenario.

Special situations can be found around the start of a steel lined underground penstock and around concrete plugs for the sealing off of an access tunnel to a pressure tunnel. Desilting chambers experience frequent water head changes, when switching between operation and emptying for sediment removal. If there are parallel chambers, these are located very close to each other and the pressure gradient from a water filled chamber to an empty one can be very high.

Even more of a special nature are compressed air surge chambers underground, gas and oil storage caverns and caverns for public utilities, civil defense and storage of goods.

Sub-sea tunnels are special in at least two respects:

The following are unlimited:
1. Salt water ingress
2. The water reservoir above the tunnel.

See Figure 3.6.

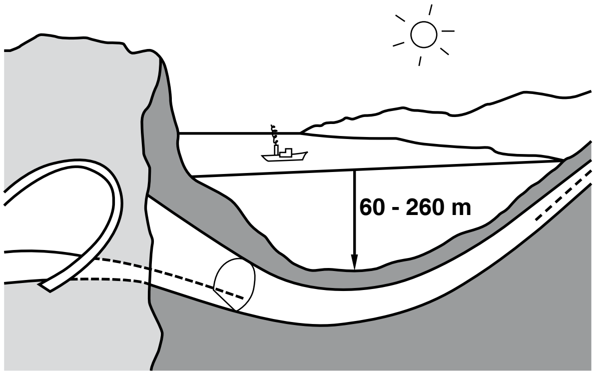


Figure 3.6 Typical layout of sub-sea road tunnel

3.5. Requirements and ground water control during construction phase

Based on the above evaluations of the functional requirements for the tunnel, the tunnel design and execution and its relation to the surroundings, a number of issues have to be decided upon regarding the ground water control program. The difficult problem to solve is how to satisfy the requirements during all stages of construction and operation of the tunnel.

One requirement that is frequently overlooked is the water ingress rate during the construction phase of the project. If the tunnel will be constructed in an urban area and ground water lowering could cause settlement damage to infrastructure on the surface, then it is not enough to plan for a final watertight permanent stage lining. It may take weeks and months between the date of exposing the ground at the face and the time when the watertight lining has been established in the same location. Meanwhile, substantial volumes of ground water may have entered the tunnel, lowering the ground water level. Frequently it is too late to prevent settlement and damage if the ground water returns to its normal level again a few months later. The situation illustrated in Figure 3.2 would be such a case.

The only available tunneling technique that can keep the ground water in-leakage near zero is the Earth Pressure Balance Machine (EPBM). It provides full face mechanical excavation using a pressurized shield and gasketed concrete segment installation with backfill grouting. Such machines are used for soil excavation and are limited to shallow depths (typically less than 30 meters).

In hard rock tunneling this alternative is not available, even if a TBM and concrete segments are used for excavation and support. Without pre-injection the leakage volume could locally become far too high. Between the time of exposure and the time of segment erection and efficient annular space backfilling, too much water could enter. With serious local water inrush at hand, such segment handling and backfill grouting would also be very difficult.

Ordinary in-situ concrete lining, even with waterstops in the construction joints, has hardly any influence on the water ingress level, as shown by ingress levels of 10 to 40 l/min per 100 m tunnel as shown in Reference [3.2]. In the Oslo area this is typically the ingress rate for an unlined and not pre-injected tunnel. Concrete lining with careful pressure grouting of the interface to the rock is still relatively successful. Concrete lining with a PVC membrane will give acceptable result, but is also not completely water tight [3.2]. Two important conclusions can be drawn:

1. A concrete lining will frequently be put in place too late to prevent permanent settlement damage on the surface.
2. Concrete lining with contact grouting or a PVC membrane will typically cost more than an extensive pre-grouting operation, achieving roughly the same final result.

Therefore, there are situations where probe drilling and pre-grouting has to be executed to meet the requirements of ground water control during the construction phase.

3.6. Measurement of water ingress to the tunnel

As described in chapter 2.3 «Design» of grouting in rock tunnels, there is no way of directly and accurately linking the grouting works effort and the final water ingress result. The result has to be monitored, corrected if necessary by carrying out post-injection as needed and by correcting the way in which the pre-grouting is being executed.
To be able to accurately determine the water ingress result after injection, it has to be measured for pre-defined tunnel lengths. Depending on the requirements and the necessary accuracy of these measurements, measuring lengths could be 10 m, 100 m or even more. The normal method of measurement is by dams in the tunnel invert (carefully prepared and sealed to avoid the wrong results by water bypassing the dams) equipped with an overflow V-notch (or any other defined shape that can be used to calculate the flow rate).

One alternative is the 90° V-notch where the height of water above the bottom of the notch can be used in the formula:

$$q = 43 \times 10^{-6} \times h^{2.5}$$

where q is flow of water in l/s and h is the water height in mm above the bottom of the V-notch. For quick reference, see the diagram in Figure 3.7.

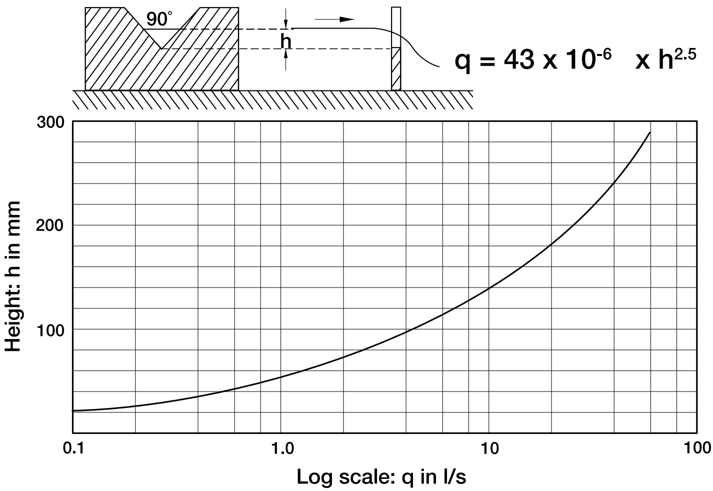


Figure 3.7 Measuring water flow rate by V-notch overflow

4. CEMENT BASED GROUTS

4.1. Basic properties of cement grouts

4.1.1. Cement particle size, fineness

Any type of cement may be used for injection purposes, but coarse cements with a relatively large particle size can only be used to fill larger openings. Two important parameters governing the permeation capability of cement are the maximum particle size and the particle size distribution. The average particle size can be expressed as the specific surface of all cement particles in a given weight of cement. The finer the grinding, the higher the specific surface or Blaine value (m^2/kg) is. For a given Blaine value, the particle size distribution may vary and the important factor is the maximum particle size, or as often expressed the d_{95}. The d_{95} gives the opening size where 95% of the particles would pass through; the remaining 5% of the particle population is larger than this dimension. The maximum particle size should be small, to avoid premature blockage of fine openings caused by jamming of the coarsest particles and filter creation in narrow spots.
The typical cement types available from most manufacturers without asking for special properties are shown in Table 4.1.

Table 4.1 Fineness of normal cement types (largest particle size 40 to 150 µm)

Cement type / specific surface	Blaine (m^2/kg)
Low heat cement for massive structures	250
Standard Portland cement (CEM 42.5)	300–350
Rapid hardening Portland cement (CEM 52.2)	400–450
Extra fine rapid hardening cement (limited availability)	550

The cements with the highest Blaine value will normally be the most expensive, due to the more elaborate grinding process.

Table 4.2 gives examples of particle size of cements commonly used for pressure injection. Please note that the actual figures are only indications based on information from the manufacturers [4.1]. There will always be some variation when testing depending on the batch.

Table 4.2 Particle size of some frequently used injection cements

Cement type	**Particle Size (μm)**	**Blaine**
Cementa Anleaggningscement	120 (d_{95}), 128 (d_{100})	300–400
Cementa Injekteringscement 64	64 (d_{95}), 128 (d_{100})	600
Cementa Injekteringscement 30	30 (d_{95}), 32 (d_{100})	1 300
RHEOCEM® 650	20 (d_{95})	650
Cementa Ultrafin cement 16	16 (d_{95}), 32 (d_{100})	800–1 200
Spinor A16	16 (d_{98})	1 200
Dyckerhof Mikrodur P-F	16 (d_{95})	1 200
RHEOCEM® 800	15 (d_{95})	820
Cementa Ultrafin cement 12	12 (d_{95}), 16 (d_{100})	2 200
RHEOCEM® 900	12 (d_{95})	875–950
Spinor A12	12 (d_{98})	1 500
Dyckerhof Mikrodur P-U	9.5 (d_{95})	1 600
Dyckerhof Mikrodur P-X	6 (d_{95})	1 900

From an injection viewpoint, these cements will have the following basic properties:

- A highly ground cement with small particle size will bind more water than a coarse cement. The risk of bleeding (water separation) in suspension created from a fine cement is therefore less, and a filled opening in the ground will stay more completely filled also after its setting.
- The finer cements will normally show quicker hydration and a higher final strength. This is normally an advantage, but also causes the disadvantage of a shorter open time in the equipment. High temperatures will increase the potential problems of the clogging up of lines and valves. The intensive mixing required for fine cements must be closely controlled to avoid heat development caused by friction in the high shear mixer and hence even quicker setting.

The finer cements will mostly give better penetration into fine cracks and openings, but one must be aware that a small number of maximum size particles may negatively dominate even if the average size is favorable.

The advantage of fine particles will only be realized as long as the mixing process is efficient enough to separate the individual particles and properly wet them. In a pure cement and water suspension, there is a tendency of particle flocculation after mixing, especially with finer cements, which is counter-productive. It is commonly said that the finest injectable crack is about 3x the maximum particle size (including the size of flocculates). For standard cements, this means openings down to about 0.3 mm while the finest micro cements may enter openings of 0.06 mm.

The question as to how one may define micro cement is often raised. Unfortunately, there is no agreed definition with an official international stamp of approval. As an informative indication of minimum requirement to apply the term micro cement, the following suggestion may be used:

- Cement with a Blaine value >600 m^2/kg and minimum 99 % of the particles having a size of <40 µm.

The above «definition» fits quite well with the International Society for Rock mechanics reference [4.2]:

- *«Superfine cement is made of the same materials as ordinary cement. It is characterized by a greater fineness (d_{95} <20µm) and an even, steep particle size distribution.»*

An example of a microfine cement that just satisfies the superfine «definition» can be found in chapter 11, MEYCO Materials. RHEOCEM 650 has a Blaine value of 650 m^2/kg and the particle size distribution shows 95 % <20µm.

The effect of water reducing admixture (or dispersing admixture) when mixing a micro cement suspension can be seen in Figure 4.1 [4.3]. It is quite evident that the reduction of d_{85} by the use of a dispersing admixture from about 9µm to 5µm will strongly influence the penetration of the suspension into the ground. If these figures are put into the soil injection criteria of Mitchell [4.4], a good injection result with this cement without admixture could be achieved in a soil with $d_{15}>0.22$ mm. With admixture the same result could be obtained in a soil with $d_{15}>0.12$ mm. Also in rock injection the effect of admixture would also be significant.

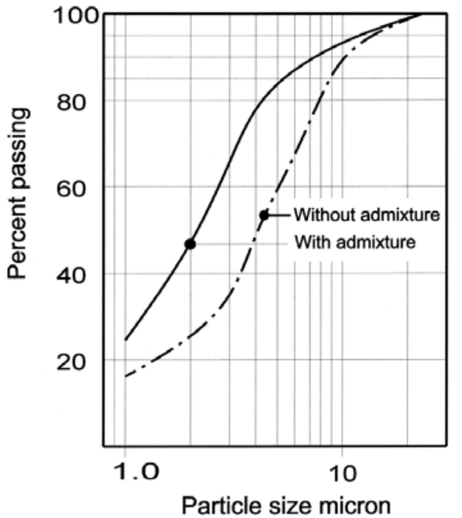


Figure 4.1 Dispersing effect of admixture when using micro cement [4.3]

Another important effect of the dispersing admixture is the lowered grout viscosity at any given w/c-ratio. The effect of lower water content is improved final strength of the grout, but more important is the lower permeability and better chemical stability. The compressive strength of a pure water and cement mix using a standard OPC is about 90 MPa at w/c-ratio of 0.3 (which will be far too stiff to be used for any normal injection work). Already at a w/c-ratio of 0.6 the strength will drop to 35 MPa and when using a w/c-ratio above 1.0 the strength is in the range of 1.0 MPa or less. (RHEOCEM 650 with 1.5% admixture and w/c-ratio of 1.0 reaches 10.0 MPa compressive strength after 28 days). More important in cases with even higher water content is that the permeability is relatively high and the strength is so low that if any water flow takes place, it can lead to chemical leaking out of hydroxides (hydration products from cement reacting with water), and eventually mechanical erosion.

4.1.2. Bentonite

Bentonite has traditionally been used on a routine basis in combination with cement for the grouting of soil and rock. The reason for this was the strong tendency of standard cement to separate when suspended in water, enhanced by the normal use of w/c-ratios up to 3.0 and more.

Bentonite can be used to reduce the bleeding in such grouts and a standard dosage of 3 to 5% of the cement weight has a strong stabilizing effect, primarily by delaying the bleeding process.

Bentonite is a natural clay from volcanic ashes and its main mineral is montmorillonite. There are two main types:

- Sodium-Bentonite (Na-)
- Calcium-Bentonite (Ca-)

Sodium Bentonite is mostly used as an additive in cement grouts, because it swells to between 10 and 25 times its original dry volume when mixed with water. The particles resemble the shape of playing cards and will adsorb water on the particle surfaces, thus stabilizing the grout mix. The particles also sink very slowly within the suspension due to their shape (see Figure 4.2).

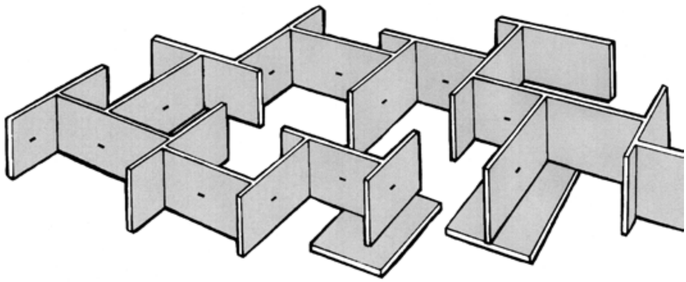


Figure 4.2 Idealized structure of Bentonite clay after dispersion in water

With the traditional cement grouting methods and materials, Bentonite had its place. However, in combination with micro cement it is normally not necessary and will be of a disadvantage in most cases. One reason is that a typical d_{95} particle size of Bentonite clay is around 60 µm. This is two to three times larger than what is found in good micro cements and will reduce the penetration achievable by micro cement. The shape of the particles is also a negative property in this respect. Modern micro cement grouts can be made with very low viscosity and limited or no bleeding if combined with chemical admixtures, and the use of Bentonite is therefore unnecessary and negative for the result.

The final strength of the grout is not particularly important in most cases involving only ground water control. However, at a high ground water head, or when a ground stabilization effect is valuable, the use of Bentonite at a normal dosage will reduce the grout strength by 50% and more. In cases where water bearing channels are large, the strength may become very important. The modern systems of micro cement and admixture have none of these disadvantages and will still deliver low viscosity and stable grouts with good penetration capability.

4.1.3. Rheological behavior of cement grouts

Cement mixed with water is an unstable suspension or a stable paste (in terms of water separation) and behaves according to Bingham's Law. Water and true liquids have a flow behavior according to Newton's Law. These laws are as follows (see Figure 4.3):

Bingham's Law:	$\tau = c + \acute{\eta}\, dv/dx$
Newton's Law:	$\tau = \acute{\eta}\, dv/dx$
where	τ = flow shear resistance (Pa)
	η = viscosity (Pa s)
	dv/dx = shear velocity (s^{-1})
	c = cohesion (Pa)

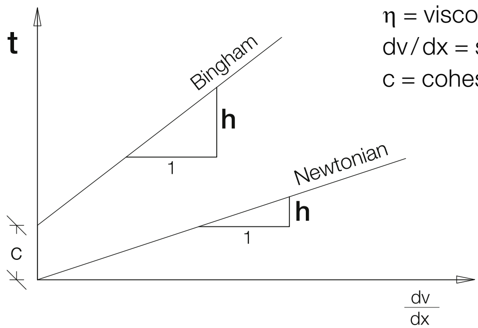


Figure 4.3 Rheological behavior of Newton and Bingham fluids

When a stable grout has a very low w/c-ratio, or when ground mineral powder or fine sand has been added, the grout may also have an internal friction. To cover this property, Lombardi proposes the following rheological formula [4.5]:

Lombardi: $\tau = c + \acute{\eta}\, dv/dx + p \tan\phi$

Where: p = internal pressure within the grout
ϕ = angle of internal friction of the grout

A true liquid will flow as soon as there is a force creating a shear stress. Water in a pipe will start flowing as soon as there is an inclination. Liquids that have higher viscosity than water will also flow but at a lower velocity.

A cement suspension or a paste will demonstrate some cohesion. The difference to liquids is that the cohesion has to be overcome for any flow to be initiated. If the internal friction is negligible, the paste will thereafter behave in a similar manner as a liquid with the same viscosity. The rheological parameters of cement suspensions can be influenced by the w/c-ratio, by chemical admixtures, by Bentonite clay and by other mineral fillers. As an example, it is possible and often useful, to create a grout with a high degree of thixotropy. This means a paste with low total flow resistance while being stirred or pumped, but shortly after being left undisturbed, it shows a very high cohesion.

4.1.4. Pressure stability of cement grouts

For the purpose of controlling grout flow in the ground and to be able to place the grout in the required place, the control of rheological parameters of the grout is vital. In this context, there is one more factor that is very important: The grout stability under pressure, which is not tested or reflected by the normal check on bleeding. The best way to illustrate the point is to consider two different grouts, both having a w/c-ratio low enough for zero bleeding. If these grouts are filled into a container with a 45 µm micro filter at the bottom and are subjected to pressure, two things may happen:

1. A grout with good stability will lose a very small quantity of water through the filter, and the thickness of dried and compacted grout on top of the filter will be very low. The main part of the grout under pressure remains uninfluenced.
2. A grout with poor stability will lose much more water through the filter over the same time, and a thick layer of dried out and compacted grout will be found on top of the filter. If the pressure is high enough

and the grout stability is very poor, all the grout volume may be dried out and compacted.

The standard method for testing the pressure filtration coefficient (K_{pf}) is the American Petroleum Institute (API) Recommended Practice 13. The coefficient is defined as the volume of water lost using the API filter press divided by the initial sample volume, divided by the square root of the filtration time in minutes, using a 6.9 bar pressure (100 psi).

Unstable mixes will typically give K_{pf} –values around 1.0, while stable mixes normally show K_{pf} around 0.1 and lower.

When squeezing out a small quantity of water from the grout at the injection front (which is well simulated by the API pressure filtration test), internal friction will quickly increase the flow resistance enough to stop further permeation. This will cause the pump pressure to increase, having the effect of more water being pressed out and the rapid development of a plug. This will happen more readily with a poor stability grout and often in positions where the openings are more than large enough to be groutable (3 x maximum particle size, as a rule of thumb).

Practical project experience and results support the above views, and it is likely that the grout stability is much more important for the permeation of a cement grout than some limited difference in particle size.

4.1.5. Use of high injection pressure

High injection pressure has proven very successful in achieving low water ingress levels, far better than what was within reach some years ago. As described above, the pressure filtration is an important factor, and it is clear that the best effect will be reached with a combination of a high grouting pressure (above 50 bar) and a grout with a low filtration coefficient.

Furthermore, the grouting pressure, when high enough, will dilate the cracks and joints of the rock formation and thus increase penetration by increasing the opening size. If high pressure is used without careful consideration of the local situation, it is possible to cause damage in the surroundings. One must be especially careful not to use very high pressure in combination with very large grout quantities in a single continuous pumping sequence.

Keil et al [4.6] used stable microfine cement injected into a granite formation. This full scale injection test was well instrumented and revealed opening and closure of fracture zones by as much as 100 μm. Analysis of specimens from the grouted formation revealed penetration into cracks as a fine as 20 μm. It should be noted that the grout used had a relatively high viscosity of 44 seconds Marsh cone time.

4.1.6. Grout setting characteristics

Ordinary Portland cements will typically show the following ranges of initial and final setting times and 24 hours uniaxial compressive strength (ISO mortar test, w/c – ratio 0.35 ± 0.05):

Initial set: 140 to 240 minutes
Final set: 190 to 240 minutes (10 to 20 MPa at 24 h)

A typical high early strength (rapid hardening) Portland cement in comparison:

Initial set: 80 to 180 minutes
Final set: 150 to 240 minutes (15 to 30 MPa at 24 h)

From a practical point of view, initial setting time cannot be made much shorter without potential problems of build-up in the equipment and clogging of material lines. It is possible to use admixtures to control the open time, which is covered separately.

One has to be aware that final set has limited relevance compared to strength or hardening created by cement hydration. Under field conditions in tunnel injection, the ground water hydrostatic pressure may be in the range of 10 to 50 bar (sometimes even higher). If a high static pressure is combined with fairly large openings, sufficient time has to be allowed at the end of an injection stage before any drilling or blasting into the same area. Otherwise, a puncture may occur and the injected cement and water is flushed back into the tunnel, thus destroying the work carried out and creating a hazardous condition. The necessary time will depend on the w/c-ratio of the injected grout, and at w/c-ratios substantially above 1.0, whether or not compaction has been carried out by a standing end-pressure.

In extreme cases, the necessary waiting time may be as long as 24 hours (above 30 bar and openings larger than 50 mm). In a more moderate case (pressure of 5 to 25 bar and maximum openings of up to 25 mm) waiting time in the range of 10 to 15 hours should be sufficient. As a rule of thumb, one must keep in mind that the compressive strength reached at a w/c-ratio of 1.0 will be only 25 to 30 % of that at a w/c-ratio of 0.4 and a further reduction to only 5 % at a w/c-ratio of 2.0. It is therefore worth carefully evaluating the situation under difficult conditions and using a lowest possible w/c-ratio.

4.2. Durability of cement injection in rock

There are many examples of hard rock tunneling with extensive use of pre-injection as part of tunnel design, and as the sole measurement of permanent ground water control. The primary experience basis is probably in Scandinavia, where Norway alone has more than 100 km of subsea tunneling with pre-grouting. Even though some of these tunnels reach to as much as 260 m below sea level and the grout injection works carried out are of a permanent nature, there is no report indicating that grout has degraded.

The Norwegian Public Roads Administration operates 20 sub-sea tunnels of various different ages (the oldest tunnel goes to Vardoe island and was commissioned in 1981) excavated through quite variable ground conditions. In fact, the general trend reported is a slow reduction of water ingress over the years. If any such degradation of grout would be a problem, one would expect an increase in water ingress levels. Melby [4.7] presents a paper dealing with 17 different projects totaling 58.6 km of tunneling. A comparison of water ingress at the time of opening and measurements made in 1996 shows the average in 1996 to be only 62.9 % of the ingress recorded when the tunnels opened. None of these tunnels showed an increase in the leakage rate.

The Norwegian national oil company Statoil constructed three pipeline sub-sea tunnels amounting to a total of 12 km, descending to 180 m below sea level. The tunnels cross Karmsund, Foerdesfjord and Foerlandsfjord. During more than 15 years of operation Statoil has recorded the energy consumption expended in the pumping of ingress water from the deepest point in the tunnels to sea level discharge. Statoil states that there has been no increase in ground water ingress, as the energy consumed for pumping has not increased [4.8].

The w/c-ratio is very important for the quality and durability of cement grouts, as well as whether the grout is stable or segregating (bleeding). Modern grouting technology in tunneling means stable grouts and a w/c-ratio below a certain limit, depending on the type of cement and the admixture used. This view is supported by the ISRM, Commission on Rock Grouting, Final Report, which states in chapter 4.2.6 [4.2]:

«Stable or almost stable suspensions contain far less excess water than unstable ones. Hence, grouts with a low water content offer the following advantages:

- During grouting:

  - *higher density, hence better removal of joint water and less mixing at the grouting front*
  - *almost complete filling of joints, including branches*
  - *the reach and the volume of grout can be closely delineated*
  - *grouting time is shortened because little excess water has to be expelled*
  - *the risk is reduced that expelled water will damage the partially set grout*

- After hardening:

- greater strength
- lower permeability
- better adhesion to joint walls
- better durability»

4.3. Controlled accelerated setting of microcement grouts

It is frequently claimed that there is no need for relatively fast setting cement for rock injection, because it is possible to use an accelerator to compensate for slow cements when needed. This is partly correct, but when considering the use of microfine cement in combination with quick setting to speed up tunneling, then the picture is different.

The main point is that accelerators will cause flocculation of the cement particles when added to the grout. In the case of a micro cement, this defeats the purpose of paying for and using a microfine cement. Furthermore, in most cases part of the injection will be carried out without an accelerator, and the accelerator is only added for special local purposes. The downside of this is that at the end of injection there will be some grout which sets fast, but most of the volume is slow and this volume will dictate the waiting time.

The MEYCO range of specially adapted micro cements for tunneling are named RHEOCEM. These cements are fast setting without any added accelerator, thus overcoming the above described problems.

Even when using a quick setting RHEOCEM grout, there are situations where accelerated setting can be necessary. This will typically be in post-grouting cases for backflow cut off, but backflow may also occur through the face in pre-injection. If for any reason the grout is pumped into running water, or pressure or channel sizes are extreme, accelerated grout may become necessary.

The most important aspect of accelerators is that they do not create flocculation before setting is initiated, and that they set rather fast. Accelerators can be diluted by adding 50% water before using them with the grout. A proven option in combination with RHEOCEM micro cement is MEYCO SA 162 accelerator (MEYCO alkali free accelerator for sprayed concrete). The practical way of using accelerators is described in chapter 9.

Normal dosage (calculated on the weight of the undiluted product) will be in the range of 0.1 to about 3.0% by weight of cement. Low dosages can be added to the grout in the agitator (although this is not recommended), while higher dosages must come through a separate hose to the packer head.

The non-return valve that is needed for use with a dosage pump for MEYCO SA accelerator through a separate hose to the packer head is shown in Figure 4.4.

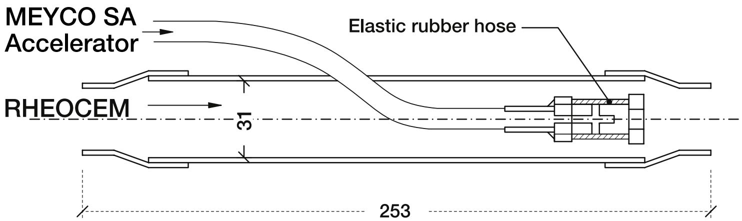


Figure 4.4 Non-return valve for accelerator dosage (dimensions in mm)

Practical experience has shown that this system works very well and can compete with other alternatives for backflow cut-off (such as quick-foaming polyurethane), usually without losing the borehole for further injection without an accelerator. It should be noted that this very practical and efficient solution does not work very well with slower cements and blended cements.

5. CHEMICAL GROUTS

Chemical grouts consist of liquid-only components which lead to quite different behavior compared to cementitious grouts. Chemical grouts are Newtonian fluids, demonstrating viscosity but no cohesion (see Figure 4.3). Therefore the penetration distance from a borehole and the placement time for a given volume only depend on the viscosity of the liquid grout and the injection pressure used. Chemical grouts available include silicates, phenolic resins, lignosulphonates, acrylamides and acrylics, sodium carbomethylcellulose, amino resins, epoxy resins, polyurethanes and polyurea silicates.

For practical purposes there are two main groups of chemical grouts available:

1. Reactive polymeric resins
2. Water rich gels

The reactive resins may be monomers or polymers that are mixed to create a reaction (polymerization) to a stable three-dimensional polymeric product. When short reaction times are used, such products are normally injected as two-component materials, with mixing is taking place at the mixing head in front of the injection packer upon entry into the ground. At longer reaction times even two-component materials can be injected by a one-component pump. The open time before polymerization and the viscosity decide about the equipment, the injection depth and the penetration capacity of reactive injection materials. Such products will not be dissolved in water, but they may react with water. For proper reaction and quality of the end product the right proportioning of the components is important. Two-component pumps must function properly at all times for this requirement to be satisfied.

The gel forming products are dissolved in water in low concentrations and the liquid components therefore show a very low viscosity (often almost as fluid as water). When the polymerization takes place, an open three dimensional molecular grid is created, which binds a lot of water to the gel. The water is not chemically linked to the polymeric grid, but is locked within the grid by absorption.

5.1. Polyurethane resins

5.1.1. General

Polyurethane resins (PU) are reactive plastic polymers that have a wide range of properties for various practical applications. Polymers are giant molecules that are produced by joining smaller molecules (monomers) in so called step growth polymerization into e.g. polyurethane products. Products with repeating units of $NHCO_2$ are called polyurethane (PU). A simple example reaction as shown below:

CH_3-N=C=O + HO-CH_2-CH_3	=>	CH_3-NH-CO-O-CH_2-CH_3
Isocyanate + Alcohol	=>	Urethane

The products may be rigid, soft, pore free or foam up to 30 times the volume of the liquid components, and the reaction time may vary between seconds and hours. The viscosity and the speed of reaction are both temperature sensitive. There are products for practical application to be injected as a single component, as well as two-component products. The properties of a product are mainly governed by the choice of different basic raw materials. Most systems can be modified by the use of added catalysts and other chemicals that influence the behavior of the product.

The very wide range of possible PU-grout properties offers an advantage to the specialist, allowing the tailoring of a material for specific purposes. For most other end users this complexity can be quite frustrating, because it will be difficult to establish which commercial product is the best one for ones intended application. The recommendations of a specialist should be sought.

In order to provide some flexibility without complicating matters too much, manufacturers will offer a limited number of standard products with a set of properties for a range of typical situations. On the basis of such a palette of standard products, it will be possible to adapt to and deal with special problems by adding extra components to modify properties.

The polyurethanes are formed by a reaction of two components:

1. Polyisocyanate (Diphenylmethane-diisocyanate, or abbreviated «MDI»). There are also other isocyanates available, but those are more hazardous and should not be used for injecting in any underground project.
2. Polyalcohols (abbreviated «polyol»).

The structure of PU molecules created from polyglycol is:

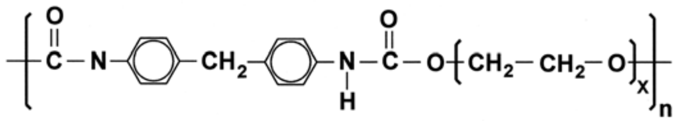


Figure 5.1 Polyurethane molecule

One very interesting part of the reaction is the effect of water. If water is added to the polyol component, or the mixed components come into contact with water after injection, a part of the isocyanate will react, producing polyurea and carbon dioxide (CO_2). This reaction takes place parallel to the formation of PU and the gas generates trapped bubbles, causing the formation of cellular foams.

In most cases underground there is a need of combined effects from an injection, such as water cut-off and ground consolidation. The cost of materials is also important. The best consolidation is reached when there is almost no foam reaction. At the other extreme, very quick foaming several times the original volume, which will produce a low strength grout that may be very effective for an initial cut-off of running water, but with little consolidation effect. A very porous grout will also not seal completely and subsequent water pressure build-up may compress the foam and increase the leakage again. The volume cost drops with increasing foam factor. The foam formation has the effect of self-expansion of the PU-grout, because the CO_2 pressure developed during the chemical reaction can reach up to 20 bar (temperature dependent). The penetration of the grout is therefore not only governed by pump pressure, by product viscosity, but is also influenced by the foaming pressure.

The properties of the foam created will depend on the local conditions. When free foaming produces a volume increase of 3 times the original size, the restricted volume increase in the ground will create a counter pressure and less expansion. A typical average volume increase in rock injection at low pressure is more likely to be 1,5 to 2,5 times.

Polyurethane products typically have a high viscosity, which is a limiting factor for permeation into the ground. At room temperature a typical product viscosity is 200 mPas (cP), but it is possible to get as low as 100 mPas (cP). If the products are diluted by the addition of solvents it is possible to come down to about 20 mPas (cP), but solvents can cause health problems and environmental problems underground.

5.1.2. MEYCO® PU-Products

See chapter 12 about various MEYCO products.

5.1.3. Pumping equipment

For two-component PU products it is necessary to use a custom designed two-component PU pump. These are normally prepared in a 1:1 ratio of the A and B components (by volume). One of the most popular pumps available for underground use (Maximator GX 45) can be seen in 5.2.

The whole set up all the way up to the packer is shown in diagram form in Figure 5.2.1.

Sequence of operation

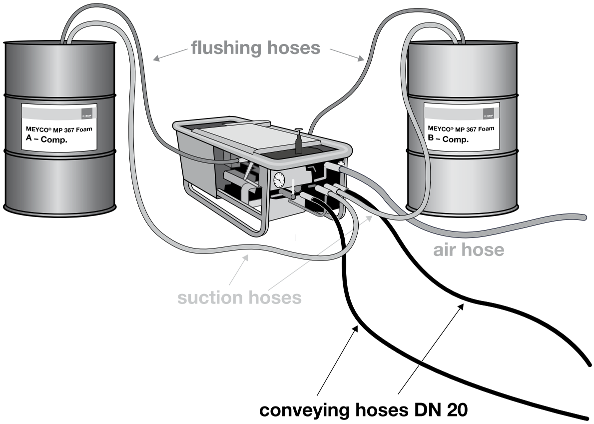


Figure 5.2 Air-driven 2-component PU pump

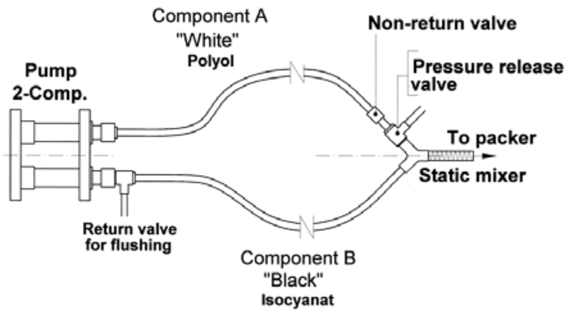


Figure 5.2.1 System diagram for 2-component injection

5.2. Silicate grouts

Sodium silicate has been used for decades, mostly in soil injection. There are also examples of silicate injection in rock formations. The main advantage of silicate grouts is their low viscosity and low cost. It may also be added that apart from the pH of typically 10.5 to 11.5 (causing it to be quite aggressive) there are small problems with working safety and health. Furthermore, this grout is questionable in relation to the environment, as the gel is generally unstable and will dissolve over time.

Liquid silicate (also called waterglass) is produced by dissolving vitreous silicate in water at a high temperature (900°C) and high pressure. The liquid is later diluted with water to reach a viscosity level that can be used for injection purposes in soil and fine cracks in rock. A normal injection grout will have viscosity of about 5 mPas (cP) and the gel produced is water rich, weak and somewhat unstable. Some syneresis will take place after the gel has formed in the ground (release of water from the gel causing some shrinkage). Because of the low gel strength it will have limited resistance to ground water pressure, especially in cracks and joints that are relatively large. This can be seen in rock injection locally, where channels may be centimeters wide, through slow extrusion of gel as pressure builds up.

The liquid silicate needs a hardener to create a gel. Acids and acidic salts will cause such a gel-formation (such as sodium bicarbonate and sodium aluminate), but today, proprietary chemical systems will normally be used, showing much better practical properties with improved quality of the final grout. These products are mostly methyl and ethyl di-esters.

If the grouting is done as ground water control of a permanent nature (several years), then silicates cannot be used. The syneresis is one of the problems in such an application that can lead to new leakage channels over time, but the chemical stability is also questionable in most cases. For temporary ground water control or soil stabilization with a required duration of some months, silicate grout will be technically acceptable. In rock injection it will often be necessary to carry out cement injection as a first step to fill up the larger channels first. The high pH cement environment is very unfavorable for the durability of the silicate grout, so this is not a good combination.

5.3. MEYCO® colloidal silica

This product bears no resemblance to the chemical silicate systems described above. The colloidal silica is a unique new system with entirely new properties and can be considered safer and more environment friendly than even cement.

See chapter 12 about MEYCO injection materials.

5.4. Acrylic grouts

Acrylic grouts already came into use 50 years ago and for cost reasons the first products were based on acrylamide. The toxic properties of such products have gradually stopped them from being used any more. The last known major application was in the Swedish Hallandsasen railway tunnel, where run-off to the ground water caused pollution downstream and poisoning of livestock. However, it is not necessary to include this dangerous component (acrylamide) in an acrylic chemical grout.

Polyacrylates are gels formed in a polymerization reaction after mixing acrylic monomers with an accelerator in aqueous solution. In the construction industry, acrylic grouts are used for soil stabilization and waterproofing in rock. Polymerized polyacrylates are not dangerous to human health or the environment. In contrast, the primary substances (monomers) of certain products can be of ecological relevance before their complete polymerization. Injection materials polymerize very quickly – as a rule, within some minutes. Before the monomers completely polymerize, a considerable amount can be diluted by the ground water (especially if there is water flow), subsequently leading to contamination.

As a result of such effects in practical injection works underground, and because of the working safety of personnel, the use of products containing acrylamide (a carcinogenic nerve poison which has a cumulative effect on the human body) must be completely rejected.

Products are available that are based on methacrylic acid esters, using an accelerator of alconal amines and a catalyst of ammonium persulphate. These products are in the same class as cement regarding working safety and can be used underground, provided normal precautions are taken.

Acrylic gel materials are very useful for injection into soil and rock with predominantly fine cracks due to their low viscosity (see Figure 5.3). Normally such products are injected with less than 20% monomer concentration in water, and the product viscosity is therefore as low as 4 to 5 mPas (cP). This viscosity is kept unchanged until just before polymerization, which then happens very quickly. This is very favorable behavior under most conditions. The gel-time can typically be chosen, ranging from seconds up to an hour.

When using acrylic grouts, one must be aware of exothermic reaction and its practical effects. Testing the gel-time in a small cup (e.g. an espresso cup), it will be substantially longer than the same test carried out in a double sized coffee mug. This happens due to self-acceleration driven by heat development (10°C higher temperature will cut gel-time by about 50%). This is important when working with such products, because slow penetration through large diameter boreholes may cause premature gelling.

Figure 5.3 MEYCO® MP 301 injected into sand

The strength of the gel will primarily depend on the concentration of monomer dissolved in water, but also which catalyst system and catalyst dosage that is being used. The gel will normally be elastic like a weak rubber with compressive strength of about 1 MPa at low deformation.

If a gel sample is left in the open for some time under normal room conditions, it will lose the adsorbed water trapped within the polymer grid, shrink and become hard and rigid. If placed in water again, it will swell and regain its original properties. In underground conditions this property of an acrylic gel will seldom represent any problem, but be aware that if an unlimited number of drying/wetting cycles must be assumed, the gel will eventually disintegrate. The chemical stability and durability of acrylic gels are otherwise very good and can be considered satisfactory for permanent solutions.

5.4.1. MEYCO® acrylic products

See chapter 12 about MEYCO injection materials.

5.5. Epoxy resins

Epoxy products can have some interesting technical properties in special cases, but the cost of epoxy and the difficult handling and application are the reasons for very limited use in rock injection underground.

Epoxy resin and hardener must be mixed in exactly the right proportions for a complete polymerization to take place. Any deviation will reduce the quality of the product. The reaction is strongly exothermic and if openings are filled that are too large (width about 1 cm) the epoxy material will start boiling and again quality will be reduced. Epoxy viscosity is relatively high, unless special solvents are used.

Working safety and environmental risk are additional aspects of epoxy injection that makes the product group of marginal interest for underground rock injection.

5.5.1. Combined systems of silicate and acrylic materials

In practical grouting it is quite normal to combine different grouts during the execution of the works. This will normally consist in reaching a certain level of tightness by the use of cement and then finalized by chemical grout. However, there are also products available where different chemical systems are combined into one commercial product.

Best known is the combination of silicate and acrylic grout. The silicate component will lower the volume cost of the final product and the acrylic component will improve the chemical stability, reduce the syneresis and give a much stronger and more stable gel.

The product will be handled as a two-component material, where the hardener for the silicate is mixed into the acrylic monomer and the hardener for the acrylic grout is mixed with the silicate. When the two components are mixed during injection, there will first be a silicate gel reaction, which is then followed by acrylic gel formation to reinforce and stabilize the final gel.

The practical handling of such a system is rather complicated and the use of such products is therefore very limited and should be left to the specialists.

5.5.2. Combined system polyurea-silicates

Lately, the mining industry has started to use a combined system of polyurea-silicates instead of traditional polyurethane resins. The product is handled as a two component material, where one component is a special silicate system and the other component a special developed pre-polymer MDI-system. The two individual components are normally mixed at a volumetric ratio of 1:1. Polyurea-silicates have many advantages compared to traditional polyurethane resins such as a high fire resistance, low flammability and an extremely fast curing. The system has a very low reaction temperature, typically below 100°C and clearly below normal polyurethane systems with reaction temperatures between 130°–180°C. The system can be designed to react with or without foaming in the presence and absence of water. These systems are used for consolidation of coal, for stabilizing of fractured rock and soil, for repair of underwater structures and for cavity filling. For MEYCO polyurea-silicate products see chapter 12.

5.6. Bitumen (asphalt)

In tunnel excavation it has happened on occasion that extreme water ingress is exposed locally at the tunnel face. Such inrush can be catastrophic and will in most cases be extremely difficult to bring under control or to seal off.
It has also happened that hydropower dams expose water channels from inside the water reservoir downstream of the dam, causing severe water loss. Leakage of this sort can be more than complicated to seal off, because it is usually not an option to empty out the water reservoir. The water pressure is therefore always present and the flow rate in the channels that need to be sealed can be very high and difficult to reach.

Ordinary cement grouts in such situations are useless. The grout has no chance to set and is diluted and flushed out by the turbulent flowing water. Up to a certain limit, quick foaming polyurethane can be used for water flow cut-off, but there are situations where this will not work, especially at low temperature (slow foam reaction). From case reports it is known that a number of very innovative methods have been tried, such as cement or concrete mixed with wood cuttings, bark cuttings, cellulose materials, foam mattress cuttings etc., and with all kinds of accelerators. Frequently all these creative approaches fail to solve the problem.

As a last resort, heated liquid bitumen (asphalt) can be an alternative. The principle is to use a selected quality of bitumen (roofing grade asphalt), that heated to a sufficiently high temperature (typically 200 to 300 °C) has low viscosity, allowing easy pumping. The softening point should be around 95 to 100 °C. The pumping output must be adapted to the water flow rate, the water head and the distance from the injection point into the water stream until the downstream outlet point. However, the asphalt output may be less than 1% of the water flow rate and still be effective, even if higher output may increase the success rate. This is totally different to all sorts of cementitious grouting, where the grout flow rate must be able to displace the water flow to have any chance of avoiding wash out.

The ideal bitumen quality will rapidly change from an easily pumped fluid material to sticky, highly viscous and non-fluid asphalt at an ambient water temperature. When injected into the water stream, the bitumen will

rapidly lose its high temperature and quickly and dramatically change its rheological properties. The bitumen gets sticky, will easily build in narrow passages in the water channel and can therefore finally block the water flow.
After blockage has been achieved, it is always advisable to place some suitable cement grout to ensure a permanent and stable barrier.

At the Stewartwill Dam in Eastern Ontario, Canada, two concentrated leakage zones through the dam foundation were grouted by asphalt (combined with cement) [5.1]. The work was carried out with a full reservoir (about 6 bar water head). The first zone, grouted in 1983, yielded 13600 l/min water leakage and this was reduced by more than 90%. The other zone, grouted in 1984, was 9000 l/min and was reduced to virtually nil. It is interesting to note that both cases were executed in one day of grouting. Material consumption was 6000 l asphalt and
5.7 m^3 sand (1983) and 3370l asphalt and 2.8 m^3 sand (1984). An unsuccessful attempt in 1982 using cement and sand took 2 months and consumed 5600 bags of cement and 73 m^3 of sand.

The specialist contractor FEC Inc. carried out injection with asphalt in Pleasant Gap [5.2], near State College, PA, USA. The grouting ran over 5 shifts and successfully cut the leakage to virtually nil.

6. BOREHOLES IN ROCK

6.1. Top hammer percussive drilling

This is the common drilling method in hard rock and medium hard rock tunneling. The drill rods attach to the drilling machine using coarse threads and the energy from the hammer travels through the drill rod to the drill bit at the end. The drilling machine delivers torque for drill rod and drill bit rotation. The rotation speed is in the range of 80 to 160 rev/minute. For hole length greater than about 5 m, the drill rods are coupled. The most frequently used borehole diameter is 51 mm, but lately, 64 mm diameter has become popular. The maximum hole length is limited to about 60 m, while the practical limit for tunnel injection could be arbitrarily set at about 30 m.

Since the late seventies, the hydraulic drilling machine has completely replaced pneumatic machines. Modern hydraulic machines can penetrate at 1.5 to 2.0 m/min even in hard granitic rocks. By coupling the drill rods, it is possible to drill very long holes, but the hole deviation will limit the practical hole length for injection purposes, as mentioned above. The directional deviation depends on a number of factors, primarily the chosen equipment and practical procedures, and secondarily on the rock conditions. Holes drilled almost horizontally will show higher deviation than vertically drilled holes.

A borehole diameter of 51 mm needs a drill rod diameter of 32 mm. The outer diameter of the couplings is 36 mm. A borehole of 30 m length with standard equipment, used in jointed and variable rock and with careless drilling (meaning maximum speed drilling with high feeder pressure from the beginning), can produce an end point deviation of 5 to 10 m (17 to 34%). By starting slowly and drilling carefully with slightly reduced feeder pressure until the first drill rod length has entered into the rock, the deviation can be reduced to less than 15%. It is also possible to apply stiffeners to the first drill rod, thus further reducing the deviation. With such equipment it is realistic to achieve deviation around 5%. One disadvantage with stiffeners on the drill string is problems of ground seizing in poor zones. The risk of getting the drill string stuck in the hole is substantially increased.

Using drilling machines such as Atlas Copco COP 1838 (Figure 6.1), the drill penetration rate with 64 mm diameter drill bits is about 2 m/min. Such a diameter allows the use of drill rods with a diameter of 38 mm. The stiffness of this system is substantially improved compared to the above, and the hole deviation is around 5% without special equipment or technique. The cost is higher and the problems of proper packer sealing in poor ground and are increased at high ground water head. A 25% increase in hole diameter gives a 57% increase of axial force on the packer from the ground water or injection pressure. It also means that the cement quantity spent for simply filling the borehole volume of one 30 m injection round with 25 holes increases from 2200 kg to 3500 kg.

Figure 6.1 Atlas Copco COP 1838 hydraulic drifter machine (Photo Atlas Copco)

A popular compromise is the use of 54 mm diameter drill bits with drill rods of 35 mm and couplings with a 38 mm diameter. This is the preferred solution today for long hole probe drilling and for injection.

For the drilling of injection holes it is important that the borehole is as circular as possible and has the correct diameter. «Correct» diameter means in relation to the selected packer, and when these two factors are correct, the packer has the best possible chance to seal off the hole without leakage. From experience, it is evident that the drill bits with a (+) configuration of the carbide inserts give the best hole circularity at the least deviation (see Figure 6.2). Both button bits and bits with a (X) configuration tend to more easily produce oval shaped holes. Furthermore, the button bit will more rapidly show diameter wear and it may produce too narrow holes for the packer, long before it would otherwise be worn out.

Figure 6.2 Drill bit for good borehole roundness and small deviation (Photo Atlas Copco)

To achieve high productivity and efficiency, drilling of probe holes and injection holes of more than 5 m length will require hydraulic equipment for the handling of drill rods, including the coupling and decoupling of rods. This is available "off the shelf" from most major equipment manufacturers. It should be noted that it is also a must from a safety point of view if the ground water head is above about 5 bar (theoretically 100 kp axial force on a 51 mm diameter drill bit). At water heads above this level, all manual handling of the drill string would be very dangerous and often not even possible.

For high productivity percussive drilling the water flushing for removal of the drill cuttings is very important. When this drilling method is used for injection boreholes, flushing is probably even more important to reduce the risk of fine material clogging the joints and cracks (which will subsequently be injected). Remaining rock cuttings may also interfere with the packer's ability to seal off the hole. Typically, about 5% of the produced rock cuttings are less than 5 mm in particle size when drilling in a granitic type of rock. The quantity of fines will most likely increase in softer rocks. A secondary grinding of particles arises from the rotation of the couplers and the drill rods, as well as friction against the borehole walls. This secondary grinding and the risk of squeezing fines into cracks and joints are greatly reduced by sufficient water flushing.

6.2. Down the hole drilling machines

This technique is also a percussive drilling method, but the drilling machine works directly on the drill bit and follows the bit into the borehole. The drill rods are there for feeder pressure, rotational torque and to convey the flushing medium. Since the hammer blows are always directly on the drill bit long boreholes will not reduce the energy delivered to the drill bit. Drilling rate is therefore not much influenced by the hole length. Typical rotation speed is 10 to 60 rev/minute.

The typical borehole diameter range is 85 mm and larger. The reason for which a smaller diameter is not available is the necessity of space for the machine. See Figure 6.3.

Figure 6.3 Down the hole machines (Photo Mission / Sandvik)

For the drilling of injection holes in underground works, this method is not normally used, however in special cases it may still be considered. This drilling method may be useful if the greater hole diameter is of benefit, if a long hole with small deviation is required, or if it is necessary to use a casing for hole stabilization. When used as part of the ODEX system (Atlas Copco), it allows a steel pipe casing to be fed into the hole in parallel with the drilling. When the final depth of the hole has been reached, the drilling machine and drill bit can be withdrawn by counter-rotating the bit, which reduces the bit diameter sufficiently for retraction inside the casing. The system is rather expensive and slow, but effective for cases where it is needed.

6.3. Rotary low speed drilling

Rotary drilling consists of point crushing under the drill bit as a result of the rotation and axial thrust. The method is not efficient for hard rock, and the minimum diameter necessary makes it unsuitable for most injection drilling.

6.4. Rotary high speed core drilling

Core drilling is also a rotary drilling method, but the drill bit is a cutting tool (as opposed to crushing). The drill rods are composed of steel pipes, and the drill bit is ring shaped with diamonds for the cutting material. Feeding pressure and rotation torque is produced by the drilling machine at the hole opening. The operations are normally hydraulic, while the machine is typically powered by an electric or diesel motor.

Core drilling is not used for normal injection drilling, but for investigations ahead of the tunnel face, and for special case injection at greater hole depth. The drilling produces a core of rock material that is retrieved from the borehole for inspection and geological logging. Normal hole diameters are 45–56–66–76 and 86 mm. Hole lengths in the range of 300 to 500 m are possible. For a distance up to circa 100 m length, the drilling penetration rate will be up to 5 m/h, depending on rock conditions and equipment. The deviation will be in the range of 2 – 3% for short holes (<15 m), and around 5 % for long holes.

Core drilling tends to produce round and smooth holes, and the clogging of cracks and joints is reduced compared to percussive drilling. The cost and time needed for core drilling is still much higher than for percussive drilling and it is therefore only used in special cases.

6.5. Example for drill and blast excavation

Shafts are the same as tunnels in many respects from an injection viewpoint (except for the fact that they are vertical or very steep). The necessity to control ground water is higher than in tunnels, because water ingress very quickly creates problems at the working face (the shaft bottom). Probe drilling to detect water bearing zones before they become a real problem by open exposure in the shaft is therefore essential.

The following is an example of a procedure for systematic pre-injection ahead of tunnel or shaft faces to be excavated by drill and blast. The described approach is based on the use of the RHEOCEM micro cement system with supplementary chemical products (MEYCO® MP 355/A3 two-component quick foaming polyurethane and MEYCO® MP 320 T colloidal silica grout), and on experience of similar operations over recent years. The example would be suitable for cases with very strict requirements on allowed residual ground water ingress.

The example is a tunnel with a diameter of 5.5 m and access through a shaft of 10.7 m diameter, where both must be sealed by grouting. The average ground permeability is $k = 3 \times 10^{-5}$ m/s and the injection should reduce it to $k = 5 \times 10^{-7}$ m/s. This is a relatively strict requirement, and consequently, systematic pre-injection has to be executed, and no effort needs to be made through probe drilling to decide about the need for injection. There are many situations where requirements are less strict, so a more relaxed approach will perform adequately.

6.5.1. Drilling of injection holes

Drilling of the boreholes is done with a 51 mm drill bit, preferably using a hydraulic drilling rig for maximum efficiency and control. During drilling, any weak zones and areas containing pressurized water are registered manually and noted in a special drilling record by the shift supervisor. Drilling of injection holes must be done with water flushing of the drill bit. A suitable drilling jumbo is shown in Figure 6.4, but there are many alternatives available, as well as units for larger tunnel cross sections.

Figure 6.4 Drilling jumbo Atlas Copco Rocket Boomer (Photo Atlas Copco)

All holes generally have a length of 20 m. By drilling and injecting into a 20 m hole length and repeating the process every 10 m, the risk of exposing unsealed undetected larger leakage channels by blasting into them is close to nil. This will be important to ensure that uncontrolled ground water in-flow is avoided. By 100% overlap of the injection fans the quality of the injection work will be good.

All holes are generally drilled at a theoretical look out angle of 11° from the shaft or tunnel contour, although other orientations may be appropriate for particular situations and/or rock conditions. The number of holes is shown in Figure 6.5. Additional holes may be drilled out from the center of the shaft/tunnel if considered necessary to achieve a water tight face. See Figure 6.5, which shows the tunnel. The same set up would apply in the shaft, only that more holes would be needed (larger diameter) to achieve about the same hole spacing.

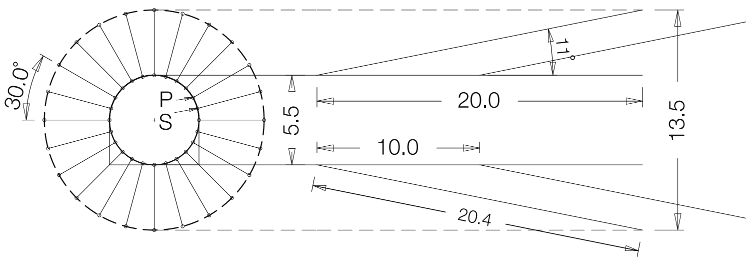


Figure 6.5 Full overlap systematic pre-injection

All holes must first be flushed with water at approximately 10 bar pressure before injection commences.

Flushing must be carried out thoroughly by introducing a stiff PVC hose to the bottom of the hole which is then slowly withdrawn while flushing. Flushing is important to get rid of all the drill sludge and fines which may otherwise block the opening of the cracks and joints (see Figure 6.8, as well as the cleaning of holes later in this chapter). In weak rock conditions flushing is not carried out due to the risk of collapsing holes, or holes where the measured ground water backflow is greater than 10 l/min from the hole.

6.5.2. Packer placement

Packers are typically placed at between 1.0 and 3.0 m depth into the borehole, adjusted to the ground conditions and locations, providing good sealing. In extremely poor ground, the grouting of standpipes made of steel or plastic may be necessary. In other cases, it may be possible to place a long inflatable packer and achieve good sealing when normal mechanical expanders cannot do the work.

In holes that yield ground water backflow, the packer should be placed as soon as possible, and the valve should be closed to minimize the ground water drainage into the excavated opening.

6.5.3. Water pressure testing

Water pressure testing of boreholes is not required as a routine activity. The long time spent in comparison to the low value of information produced is the main reason for this. When using RHEOCEM microfine cement at a fixed w/c-ratio, which can cover a wide variation in ground conditions, there is no important reason to invest time and money in such measurements.

6.5.4. Choice of injection materials

Based on typical hard rock ground conditions and a tightness requirement of less than 5 l/minute and a 100 m tunnel, RHEOCEM must be selected as the primary injection material. Boreholes yielding ground water inflow of more than 5 l/min must be cement injected in any case as well as all primary stage boreholes.

Depending on ground conditions, grouting by colloidal silica mineral grout with MEYCO® MP 320 may be necessary or beneficial as a supplement. In such a case this product should be used in the secondary stage boreholes (unless the ground water inflow in individual holes is too large, as stated above).

Inflow of ground water through joints and cracks in the face or elsewhere (outside of the boreholes), may cause problems of grout washout and backflow. Such problems can be solved by using an accelerator added to the RHEOCEM grout or by using two-component pumps and an increased dosage of accelerator in MEYCO MP 320. Separate injection of the two-component quick foaming polyurethane MEYCO MP 355/A3 is another alternative. This product reacts very quickly, and the reaction time is adjustable by adding the appropriate amount of accelerator. PU-foam can be used for an immediate and temporary flow blockage, but should always be followed up by using cement or another permanent grout.

6.5.5. Mix design for RHEOCEM® grouting

RHEOBUILD® 2000 PF admixture: 1.5 – 2.0 % by weight of cement.
Normal w / c-ratio for the whole RHEOCEM range: 1.0

The w/c-ratio of 1.0 allows a non-bleeding grout with a very low viscosity of about 32 seconds Mash Cone flow time. This grout mix design should be kept constant and only special conditions will require an adjustment. This could be, for example, contact to very large joints and channels that could benefit from a lower w/c-ratio and an even stronger set grout.

In cases of uncontrolled spread of grout or backflow to the shaft/tunnel, the MEYCO SA accelerator can be added to the RHEOCEM mix by dosage at the packer. Very short initial and final set can be achieved, which will prevent wash-out.

6.5.6. Accelerated cement grout

The MEYCO SA alkali-free accelerator can shorten curing time of the grout to minutes, which allows an early recommencement of excavation operations even at high ground water head and large fissures. One must be aware that the efficiency of this technique depends on the type of cement. Most other cements will give much longer reaction times than RHEOCEM, and in many cases it will not work as required.

As example the normal dosage of MEYCO SA 162 is between 1 – 5% by weight of cement.

MEYCO SA alkali-free accelerator must be added at the packer from a separate dosage pump, delivering the accelerator through a specially designed non-return valve. See chapter 4: Cement based grouts under Accelerators for cement injection.

6.5.7. Pump pressure

The injection pressure is important for the success of the injection and needs to be as high as conditions allow. This is one of the advantages of injection ahead of the face (compared to post-injection) and should be used fully.

As noted above, relatively high pumping pressure is generally possible with the pre-injection approach, because injection is made into undisturbed rock ahead of the tunnel face. The available pumping equipment should therefore be capable of producing controlled pressure of up to 100 bar measured at the pump.

However, situations may exist where other factors influence the choice of injection pressure and the methods of control. Some typical special measures are given below.

6.5.8. Special measures

Special measures should be adopted for injection operations where either adjacent works are very close, or the rock overburden is limited.

If the distance from the point of injection to other structures is less than 10 m, or the rock overburden is in that same range, special care must be considered. Very weak and broken ground may also call for this, even at a greater distance than 10 m. Where necessary, the following special measures can be adopted as appropriate:

- Injection pressure must always be controlled by monitoring the line pressure and by pre-setting an appropriately low pump cut-off pressure.
- The grout take per hole can be limited to less than the general maximum if the pressure increases above a defined limit. This will limit the lifting force by limiting the area under pressure and the level of grout pressure. Because of the non-bleeding property of the RHEOCEM grout and its fast gelling when pumping stops, it will remain in place and permanently fill the occupied volume without being kept under pressure over time. A grout to refusal technique for consolidation is not necessary with RHEOCEM, as there is no superfluous water to squeeze out.
- There are a number of special measures available for the protection of existing tunnel linings or other structures in close proximity to the injection point. In summary, these measures include:

  - Pre-determined pressure cut-off on pumping pressure
  - Pre-determined limit of grout volume per hole
  - Pre-determined limit of injection time per hole
  - Controlled setting time of the grout
  - Temporary pressure relief
  - Continuous visual monitoring with telephone or radio communication to the injection supervisor during critical injection works

Measures such as these have been used previously to control pre-injection operations where overburden has been as low as 4 meters (e.g. Ormen Water Tunnel, Stockholm).

6.5.9. Injection procedure

Always start the injection in the lowest hole in the face (tunnels) and progress successively towards the roof until all holes are injected.

Unless special measures are required, the injection on an individual borehole is completed when the flow rate of RHEOCEM grout into the hole is less than 3 liters/minute at the maximum allowed pressure specified, or when more than 1000 kg cement has been injected (the quantity limit will vary with project requirements and geological conditions).

In the case of colloidal silica mineral grout injection (MEYCO MP 320 T) an individual hole is finished when the flow rate is down to less than 1 liter / minute at the specified maximum pressure, or the total quantity has reached 500 kg (the quantity limit will vary with project requirements and geological conditions).

In cases where backflow leakages occur directly out of the rock surface, RHEOCEM should be used with MEYCO SA accelerator to block it or reduce the gel-time of the colloidal silica as needed. In very difficult cases, quick foaming polyurethane MEYCO MP 355/A3 should be used.

If two or more holes become inter-connected during the injection process, the valves on the packers in holes that are connected to the injection point should be closed. The amount of grout specified per hole should be multiplied by the number of holes connected and pumped into all of them before regarding the grouting as completed.

6.5.10. Injection records

During injection the following parameters should be accurately noted in the Injection Record (electronic recording can be a major help in this respect):

- All necessary and relevant general information about the project
- Ground water flow from the holes
- Injection materials and mix design
- Pressure at the beginning and at the end of each injection, including grout flow rates
- Injection time per hole
- Total material consumption per hole
- Number of holes, stage of grouting
- Any surface leakages and backflow
- Grout hole inter-connections

6.5.11. Cement hydration – waiting time

Packers can be removed from the holes 1.5 – 2.0 hours after completion of the injection and drilling of secondary stage holes (or drill and blast holes) can start 2 hours after completion of injection with RHEOCEM.

Where injection has been done with grout containing MEYCO SA accelerator or with a w/c-ratio of <0.7 packers can be removed from the holes approximately 40 – 60 minutes after completion of the injection. Drilling of new holes in this case can start 60 minutes after completion of the injection.

Be aware that the time frames mentioned above may be 5 to 15 times longer when using traditional techniques with most other types of cement.

In the case of colloidal silica grout MEYCO MP 320, a minimum of two times the gel time should be allowed from the completion of injection until the next drilling starts.

6.5.12. Other relevant issues

All pre-injection works must be carried out under the supervision of a specialist with relevant qualifications and experience.

The Contractor must submit a complete Method Statement (MS) for the pre-injection works before they begin. The MS must contain information about, but not be limited to, the following:

- Listing of all personnel and supervision with clear description of responsibilities and authorities
- Drilling and flushing equipment and methods
- Type of packers and valves
- Material, mix designs and quality control procedures
- Proposed site trials
- Injection plant presentation
- Injection procedures and forms of record keeping
- Integration of systematic pre-injection into the construction cycle

- EHS management and control

All pipes, hoses and connections are designed to withstand the maximum pressure capacity of the pumping equipment and are drip-free when in operation.

The mixer for cement grout should be a high shear colloidal mixer with a rotor speed of minimum 1500 rpm. After the mixer a holding tank should be provided, equipped with an agitator slowly running at all times. The pump should be a duplex piston type pump operated hydraulically to allow the pre-setting of the grout flow and the maximum pumping pressure.

Holes injected by MEYCO MP 320 should be filled by cement grout from the packer location to the opening. This grout should be of a low w/c-ratio type suitable for grouted bolts.

6.6. Example solution: hard rock TBM excavation

Hard rock TBMs are open machine layouts with just a part roof shield over the front of the machine. The system is designed for fast advance and the economy of the project primarily depends on the rate of advance. Hard rock conditions will usually demonstrate stable and good ground for a major part of the tunnel. However, short sections through crushed shear zones with clay gouge material may cause serious time delays. Spiling rock bolting is very efficient under such circumstances, provided the fully grouted rebar bolts can be placed efficiently, which requires proper drilling equipment.

Environmental restrictions may also influence TBM tunnel excavation projects. Even such tunneling may require strict ground water control and limits on water ingress, because of the normal potential consequences of ground water lowering above the tunnel.

To be able to take advantage of spiling rock bolts and to execute pre-injection, it is an obvious prerequisite that the necessary boreholes can be placed in the right positions and at the right angle. In drill and blast excavation this is simple, but in TBM projects it has repeatedly turned out to be difficult. More than once owners have accepted bids containing reassurance from the contractor that the drilling method will be sorted out later. Experience shows that if “later” means after the start of the TBM operation, it is usually too late. The simple fact is that the TBM has to be designed and adapted for the purpose of pre-drilling and the equipment must be installed before the TBM goes underground.

6.6.1. The Oslo Sewage Tunnel System

The tunnel system consists of about 40 km of sewage transport tunnels and an underground sewage treatment plant. Construction was undertaken around 1980. The tunnels were constructed by TBM to avoid vibration problems when passing below urban areas. Pre-injection was mandatory because a major part of the buildings and infrastructure are founded on marine clay. Even a minor lowering of the pore pressure in the clay basins would cause settlements up to several hundred meters away from the tunnel alignment.

The first contract let was based on a 3.5 m diameter hard rock Robbins TBM. The contractor stated that the probe drilling and injection drilling would be solved «later». The method finally adopted was very poor. The TBM was reversed a couple of meters, people and equipment were passed through the cutter head and drilling was carried out by manually operated pneumatic jackleg drills. This system was grossly unsatisfactory and caused the owner to reject all bids for subsequent project sections, if the detailed pre-injection drilling solution was not included and considered acceptable.

The largest single tunneling contract covered 14.2 km of 3.5 m diameter TBM tunneling and a 900 m drill and blast access tunnel at Holmen. The two Robbins TBMs were manufactured to accommodate two hydraulic booms each with Montabert H70 hydraulic bore hammers. The feeder length was 3 m (10 feet).

During planning of the equipment setup it was found that all components had to be adapted to integrate with each other. This adaptation included the TBM. The final outcome provided 17 locations around the periphery where boreholes could be started. From each location drilling could be carried out in variable directions. The starting points to collar the holes were 3 m behind the face.

The equipment layout can be seen in Figure 6.6. The normal borehole angle relative to the tunnel contour was 4°. By drilling 27 m boreholes, losing 3 m between the starting point and the actual face and 4 m overlap to the next drilling, a net length of 20 m of tunnel was pre-injected per round. This is shown in Figure 6.7.

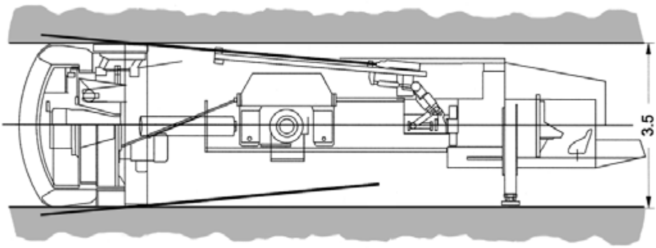


Figure 6.6 Tailor made hydraulic drilling equipment mounted on hard rock TBM

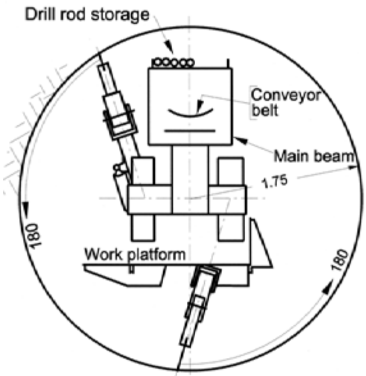


Figure 6.6.1 TBM cross section with drilling equipment

Work in the tunnel was organized in two shifts per day, each shift being 7.5 hours. The routine drilling of four 27 m long holes normally required 3 to 4 hours including setup and clearing away. The injection time was highly variable depending on the quantities injected. It is therefore no surprise that the weekly face progress also varied accordingly. The average weekly advance was about 60 meters.

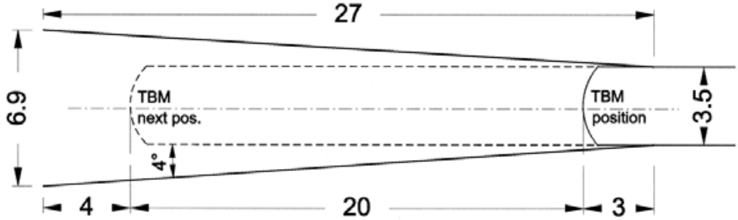


Figure 6.7 Borehole length and net coverage per grouting stage, plan view

The total cost of pre-injection, including drilling ahead, was 38% of the total contractor's cost per meter of tunnel.

6.6.2. The Hong Kong Sewage Tunnel System

More than 1.5 million m^3 of waste water is discharged into Victoria Harbour per day, and to reduce pollution the government of Hong Kong decided to construct a deep sewage collection and conveyance system with a total length of about 70 km. Stage 1 includes 23.6 km of tunnels from the NE areas of Hong Kong Island and Kowloon to Stonecutters Island. A treatment plant has been built to purify the sewage before disposal via a submarine outfall to the western approach of Victoria Harbour.

Construction of the Stage 1 works started in April 1994. The deep tunnel conveyance system and the outfall are 2.2 to 5.0 m in diameter, running at depths between 76 m and 150 m below sea level and were excavated by TBM.

Skanska International Civil Engineering AB was the main contractor for contract DC/96/20 including 3580 m of TBM tunnel. Grouting was carried out to limit inflow of water for safe tunnel boring and to enable permanent concrete lining to follow. The aim was met by pre-grouting supplemented by post-grouting where needed.
The method reached after a long period of optimization was to execute pre-grouting when probe holes yielded 2 l/min/m. Key points in the procedure were:

- Stable grout (<5% bleeding)

- Micro cement with admixture to allow low viscosity (RHEOCEM 650)
- Optimized drilling pattern

(As illustrated earlier, the TBM must be prepared and adapted for pre-injection before going underground. In Oslo, there was an earlier attempt at constructing these tunnels that failed because this was not done).

6.6.3. Comments on drilling and injection equipment

Drilling ahead of a hard rock TBM is difficult because of the very limited available space close to the tunnel face. The TBM itself occupies almost all of the volume for about the first 15 m of the tunnel.

The solution showed from the Holmen Site was established after elaborate design work, including the TBM manufacturer's design staff. The main beam of the TBM was elongated by about one meter to accommodate the two hydraulic booms. Even with this solution, the starting points for probe holes and injection holes are about 3 m behind the face. This distance is critical to the performance of the total setup.

There are examples of starting points at greater distances such as 6 m (Skjeggedal [6.1]) and 9 m from the face. The last figure was given by Avery [6.2] from the Inland Feeder Project, Arrowhead East tunnel, San Bernardino, California, where they finally decided to drill through the cutter head itself. In the Hong Kong sewage project more than 10 m of distance was tried but later improved (reduced) as a matter of necessity. It is obvious that the starting point distance multiplied by the number of holes gives the meters drilled and wasted per round of drilling. Lost time and additional cost becomes critical with increasing starting point distance.

Injection often requires pumping pressures of more than 30 bar. Because the drilled holes are started in the wall at 4° to 8° in most cases, the distance to the excavated tunnel wall is very short. To avoid a sideways blowout and grout backflow into the tunnel, the only safe solution is packer placement in front of the face location. This may require packer placement depth of more than 10 m on a routine basis, which again is very time consuming and expensive.

6.7. Cleaning of injection holes

Holes drilled for injection of grout must be cleaned properly. The effect of not doing so can already lead to rapid blockage of the intersected water bearing channels in the first few mm of the channel, measured from the borehole wall. This can happen when sludge and rock cuttings from the drilling process are forced into openings by the injection material and the pumping pressure. The effort spent in drilling the borehole may, in the worst case, be more or less wasted.

An investigation carried out in Norway in 1982 and reported at the yearly Norwegian Rock Blasting Conference gives an indication of the importance of cleaning. Unfortunately, the amount of data available from tests is low and therefore not conclusive. However, the results are fully in line with practical experience from injection works in rock.

A custom designed diesel powered piece of equipment is used to provide a water jet pressure of 250 bar. The water is pumped through a high pressure hose to a nozzle with some water jets pointing 45° back along the hose and others at 90° radially. With this configuration, the nozzle is self-propelled forward into the hole and can be removed by just pulling the flexible hose.

The pump is also used for water pressure testing (WPT) and to execute hydraulic splitting of the boreholes (if no water take is measured at the normal 10 bar testing pressure), by providing water pressure up to 350 bar. The equipment diagram is shown in Figure 6.8.

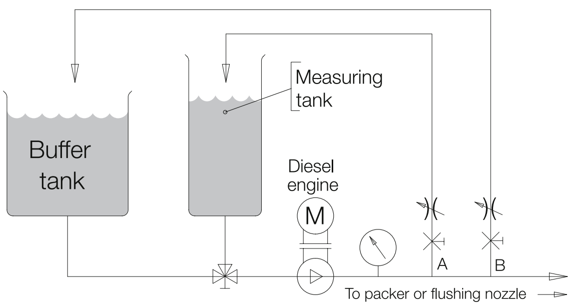


Figure 6.8 Equipment for borehole flushing, WPT and hydraulic fracturing

For the cleaning of boreholes and for hydraulic fracturing, the pump draws water from the buffer tank. When executing WPT, the valve close to the buffer tank for water supply only draws from the graded measuring tank.

During high pressure cleaning of boreholes valves A and B are closed. The nozzles are designed to give 250 bar pressure at a flow rate of 40 l/min. For safety reasons the water supply hose has a foot operated valve giving free low pressure water flow when not operated. When ready to start flushing, the operator steps on the valve and all the water passes through to the flushing nozzle.

For hydraulic fracturing the foot-valve is removed, valve A closed and valve B opened. The fixed nozzle in series with valve B is designed to give 150 bar at 40 l/min if the borehole is tight. Other nozzles can be used if a higher maximum pressure is required.

For WPT valve B has to be closed and valve A opened. The nozzle in series with valve A is adjustable and can be used for adjustment of the pressure (depending on water flow into the borehole), to keep it as close as possible to 10 bar (which should be used for WPT according to the standard procedure).

The holes are cleaned first by the traditional method of water and compressed air flushing through an open flexible PVC pipe pushed to the bottom of the hole. The typical hole length is 10 to 15 m. After this normal way of cleaning, WPT is executed with the described equipment.

The next step is high pressure cleaning and the nozzle is self-propelled to the bottom of the hole and pulled all the way out and repeated three times. Then another WPT is carried out. Tight sections are subjected to hydraulic splitting to see if it is possible to create permanent connections usable for injection.

The investigation demonstrated that the equipment proved to be very practical, and the cleaning process could be carried out much faster and more efficiently than with traditional methods. The equipment was also well suited for WPT and for hydraulic splitting. In the two locations where there was water take measured after normal cleaning, the measured values increased by 8.2% and 55.3% after high pressure cleaning. In places where there was no water take, the hydraulic splitting created channels yielding up to 3.04 Lugeon. The quantity of additional dry material washed out of the already cleaned holes varied from 0.5 kg to 40 kg per hole. The maximum quantity came from a hole that intersected a clay seam.

6.8. Packers

When a hole has been drilled into the rock formation for the purpose of injecting grout at high pressure, a tight connection (seal) between the pumping hose and the borehole is needed. The common method to achieve this is to insert a so-called packer.

The packer (or expander) consists of a pipe with a coupling at the tunnel end and an elastic expander that can be inserted into the borehole and expanded against the borehole wall. The expander will anchor the packer in place so that the injection pressure is not forcing it out of the hole and it also seals off the pressurized section of the borehole from the tunnel side. The injection pump material hose is hooked up to the pipe and the pump can be started.

There are a number of different packer types available, and some examples are shown below for illustration. Most manufacturers produce the same type of packer, but they may be quite different in quality, range of expansion, dimensions and technical details. Packers must be selected for each individual project based on ground conditions, availability, price and a number of other factors.

6.8.1. Mechanical packers (expanders)

The mechanical expander is intended to be re-usable and works in principle as shown in Figure 6.9. The diameter of the rubber expander has to be in a certain relation to the borehole diameter (smaller) and the maximum expansion range provided by the packer. The manufacturers give accurate information about these details for their individual products. Normally, packers can be delivered in different standard lengths (pipe and expander assembly), typically from 1.0 m to 5.0 m in steps of 0.5 m, but the user can also produce his own pipes locally and choose any suitable length. For very deep packer placements it is normal to use connectors to join standard pipe lengths of e.g. 3.0 m.

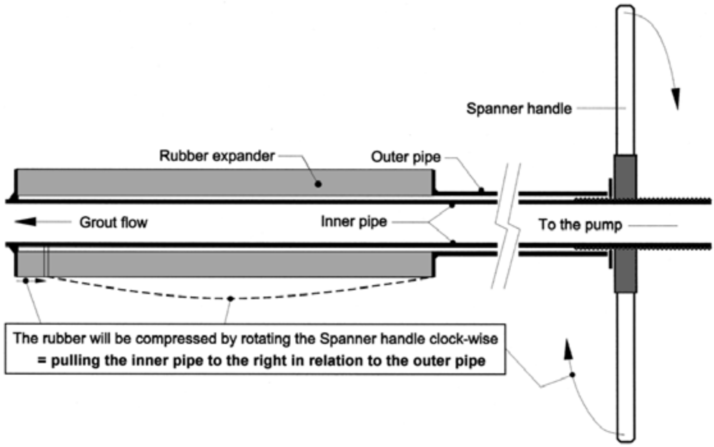


Figure 6.9 Mechanical borehole packer

At the tunnel end of the packer pipe a ball valve or a similar device is fitted. When the injection is completed, the ball valve can be closed and the pump hose disconnected (see Figure 6.10). Without the valve, pressurized grout would flow back into the tunnel. The valve must remain closed with the packer in place until the grout has set sufficiently to keep the ground water pressure without backflow. The packer may then be removed and cleaned for re-use in a different hole.

Figure 6.10 Ball valve and hook-up for the grout hose

Cleaning of packers of this type can be quite time consuming, and if they are removed too late, it may become impossible to open them. It is very important to have the right tools in place for cleaning, and solutions using high-pressure water jets seem to be the most effective. Loss of some of the packers is still quite normal, especially if fast setting grout and accelerators are being used. With polyurethane grouts it is often not feasible to carry out any cleaning. If cleaning and maintenance of the packers is not handled well, the cost of wasted packers may become quite high. The tendency has therefore been to use less of this traditional type of packer and to use more of the disposable (single-use) packers.

6.8.2. Disposable packers

For reasons given above, the disposable packers are frequently a good alternative to re-usable packers. They generally work in the same way as the re-usable ones, but are constructed so that when expanded, the expansion is automatically locked in place to allow removal of the inner and outer pipes used to place the packer and expand it. The packer itself may have a one-way valve to keep pressurized grout in place without backflow when releasing the pump pressure and removing the pipes.

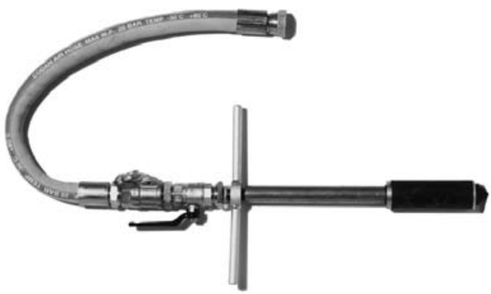


Figure 6.11 Disposable packer with installation assembly

Such packer assemblies are illustrated in Figure 6.11, and Figure 6.12 shows four different standard dimensions.

Figure 6.12 Disposable packers, 38 to 63 mm diameter (Photo Roulunds Codan)

The same types of packers with minor modifications can be used as re-usable packers by removing the expansion lock ring and the non-return valve at the tip. It is also possible to force the non-return valve to stay open when required by inserting a short piece of pipe through the valve, e.g. to be able to detect connections from other boreholes being injected or during water pressure testing.

6.8.3. Hydraulic packers

The hydraulic packers are expanded (or inflated) via high pressure water supplied through a separate thin line from the tunnel to the packer location. The packer is only handled via a single pipe, which is also the grout conveyance pipe (or hose) after packer expansion. The length of the packers (the expanding part) can vary from 300 mm to more than one meter (see 6.12.1). They have a much wider expansion range and will seal better in difficult ground (due to the expander length). However, they are also substantially more expensive, and if such packers are regularly lost due to removal after the grout has set too much, the cost will quickly become prohibitive.

Figure 6.12.1 Large inflatable packer (Photo Roulunds Codan)

For WPT in long holes, these packers are practical because they are quick to expand, deflate and move. The low risk of backflow around the packer due to good sealing properties in poor ground is also very favorable. Figure 6.13 shows an inflatable packer assembly.

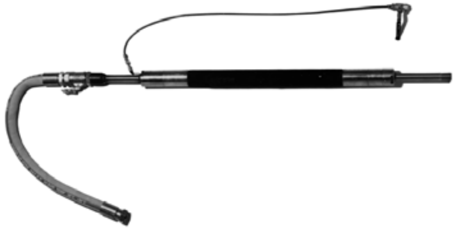


Figure 6.13 Inflatable borehole packers (Photo Roulunds Codan)

When it is necessary to execute WPT or inject shorter sections of boreholes, the hydraulic double packer can be used. See Figure 6.15. Here, two packers are coupled in tandem at a fixed distance. When expanded in the borehole, the grout will only fill the borehole between the expanders, and only this section of the hole will be subject to injection pressure (or water pressure in the WPT procedure).

The mechanical and disposable expanders may not work properly in very weak or broken ground, as it is difficult to make them seal without backflow. They may start sliding in the hole under pressure and can become stuck in the wrong position. A combination of a hydraulic packer and a disposable self-locking packer can be very useful and economic under such conditions. This special hydraulic packer allows expansion of the disposable packer which is mounted in front of it, and the hydraulic packer helps prevent backflow and sliding during injection. After a short waiting time for the grout to set, the hydraulic packer can be removed while the disposable one remains in place.

6.8.4. Standpipe techniques

There are situations where the ground is so poor that the placement of the packer is very difficult or impossible, and borehole stability may be a problem. In hard rock tunneling this occurs in crossing shear zones and highly broken ground. When such conditions are combined with high water ingress at high hydrostatic head, the combination may lead to loss of face stability and a progressive collapse.

The drilling of long holes is usually a separate problem because the drill string may easily become stuck and can break in the borehole.

If such conditions are encountered, it is important to establish a safe position to work from. This position can be created by the grouted overlap zone from the previous round of injection, or by not advancing the face from good ground into poor ground without pre-injection. Improvising with a shallow packer placement in very poor ground should be avoided, as it allows the high water pressure to attack very close to the face, due to the drilled probe or injection holes. Neglecting to act properly in such conditions may lead to a face collapse and loss of control.

One of the best ways of dealing with serious problems of this type is to use the so-called standpipe technique. Through drilling with an oversize drill bit of e.g. 76 mm diameter to a depth of circa 3 to 4 m, it is possible to insert a steel pipe of suitable diameter (i.d. >55 mm, o.d. <66 mm) into the hole. The pipe must be grouted into place using a high quality shrinkage compensated cement grout. This is easy to do by placing a packer close to the inner end of the pipe and by pumping the grout into the annular space between the pipe and the rock until it appears at the borehole collar. See Figure 6.14

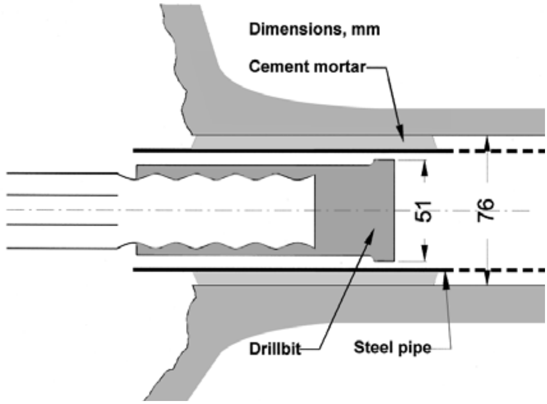


Figure 6.14 Use of a standpipe in poor unstable grounds

Placement of standpipes may become difficult if the face has been advanced too far forward due to water ingress through the face. It must be emphasized that serious delays and practical problems can be avoided by not taking any risks in this respect.

Once the grout has set around the steel pipe, the hole can be extended by an ordinary 51 mm diameter drill bit (as indicated in Figure 6.14). As soon as drilling hits serious problems of any kind, a packer may be placed safely and tightly in the steel pipe and the newly drilled part can be injected. After grout setting, the drilling can be resumed for another step of borehole extension. The process may be repeated as required.

When the boreholes are drilled from the tunnel contour at the face and angled outwards, the steel pipes will work like spiling rock bolts and improve stability quite efficiently. If the standpipes need to be drilled in the tunnel face, it is possible that steel pipes cannot be used. As an alternative, plastic pipes can be placed in the same way as described above. These do not cause problems for TBMs or roadheader excavators and can be easily broken without damaging cutters or picks.

6.8.5. Tube-a-manchet

This is a technique frequently used in soil injection (mostly steep or vertical holes), but it is not common in rock injection underground. Figure

6.15 illustrates the characteristics of this method. The figure shows a hydraulic double packer inserted into a sleeve pipe surrounded by mortar within a drilled hole. The sleeve pipe sits in a type of mortar called mantel grout, which is a simple cement grout with a relatively high content of Bentonite clay. The sleeve pipe has non-return valves (rubber sleeves) at a fixed distance, and these valves can be activated individually by injection pressure between double packers. The mantel grout is designed to be weak so that it will split when pump pressure is applied, allowing grout to flow into the ground without escaping along the borehole. The packer can be moved as needed, and each valve can be grouted several times.

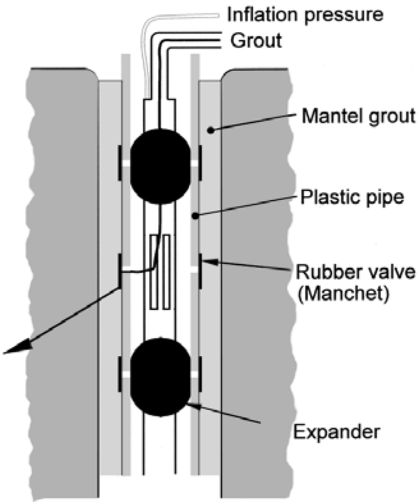


Figure 6.15 Tube-a-manchet (sleeve pipe) principle

The mantelgrout mix design is rather simple:

▪ Ordinary Portland cement	720 kg
▪ Water	1 450 kg
▪ Bentonite clay	72.5 kg

The Bentonite clay should be pre-hydrated with some of the mixing water before producing the mix.

In tunnel injection where holes are frequently sub-horizontal, it is very difficult to ensure proper filling of mantelgrout around the entire sleeve pipe, especially along the top of the hole. Grout leakage from the top of the hole to its opening may prevent the build-up of grouting pressure and penetration into the ground.

6.8.6. Drill anchors

In areas where the rock is heavily fractured and unstable, it is sometimes not possible to insert a packer due to borehole collapses. A good alternative to standpipes to solve this problem is the use of drill anchors. These drill rods are fitted with a suitable drill bit and are drilled to full depth and left permanently in the borehole. Once all drill anchors are placed, a re-usable adaptor is attached to the end of the anchor to allow the hook-up of the injection hose. The drill rod, which has been grouted in place, works like any other steel bolt, providing extra stabilization in addition to the contribution from the injected grout. These bolts are frequently named self-drilling anchors.

Due to the annular space between the steel rod and the rock wall, pressure grouting through the end of the drill rod will frequently flow back to the borehole opening at low pressure. To avoid this problem, short grouted sections may be placed along the drill rod and allowed to set, before pressure grouting the rock.

6.9. Probing ahead of the face

6.9.1. Normal approach

In tunneling it is quite normal for information concerning the details of rock conditions in front of the tunnel face to be limited and not particularly reliable. The general average conditions may be reasonably well known, but that is of little help if a local feature is suddenly exposed, yielding several thousand liters of water per minute at high pressure. Furthermore, the contrast between normal hard rock tunneling conditions and the sudden occurrence of a major shear zone containing swelling clay and crushed rock can be quite dramatic. When exposed without warning, this often magnifies problems.

Probing ahead of the tunnel face by percussive drilling is one way of reducing the risk created by not being prepared. Percussive drilling is not an optimal method for the mapping of rock conditions or the investigation of hydro-geological conditions ahead of the face. It is however, the best method available for the investigation of water features in a reasonable time and cost frame when operating from a tunnel face. A diamond core hole will produce more information and more accurate data, but it takes too much time to be used as a routine tool. Due to this fact, as well as the resulting limited number of holes, there is a risk of not detecting features of importance.

In a drill and blast tunneling operation the equipment is already present, and the additional effort of drilling some of the blasting holes to greater depth for probing ahead is minimal. A minimum probing would be one single hole, extending some distance beyond the blast holes. Two to five holes reaching 20 to 30 m from the face will usually be made. The number of probe holes that are necessary will depend on the size of the tunnel, the rock and ground water regime, and the potential consequences of not detected problems. A general rule is not available, but regarding water inrush risk, the probability of problem detection will increase proportionally with the number of holes drilled to a hole spacing of about 5 meters. Reducing risk by further reduction of the probe hole spacing will give diminishing returns.

In sub-sea tunnels, below rivers or lakes or anywhere with a high risk if the rock cover is less than expected, the probe drilling is targeted at more than just detecting water. A drilling pattern for a sub-sea two lane road tunnel has been presented by Blindheim [1.2] as shown in Figure 6.16.

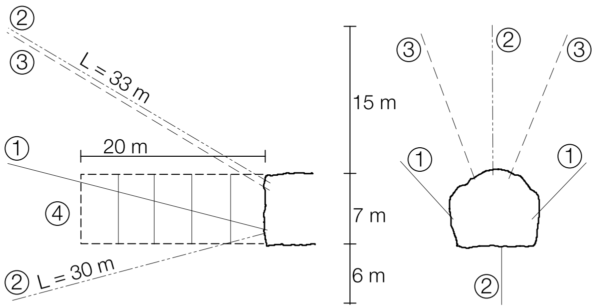


Figure 6.16 Probe drilling for a sub-sea two lane road tunnel [1.2]

1. *Routine minimum probe drilling*
2. *Additional holes in expected weakness zones*
3. *Alternative holes in sections of low rock cover*
4. *Overlap (minimum 8 m) for each 5th blasting round*

In tunnels excavated mechanically, equipment for probe drilling will be one extra unit. In roadheader excavation a small drill jumbo can be used. For any kind of full face machine (TBM), custom designed equipment must be mounted on the TBM to be effective.

Percussive drilling produces boreholes in the ground, and as a result, water and the sludge from drill cuttings will come out of the hole. A trained operator or an experienced geological engineer can log information such as changes in the drilling rate, the color of the sludge, changes in fragmentation of the sludge, loss or reduction of flushing water, sudden increase of water out of the hole etc., which are all linked to the depth from the tunnel face. The observations must be noted in a prepared log. The interpretation of the observations must be expressed in writing, but separated from the basic data. From the observations and the resulting interpretation, decisions can be made on possible action regarding additional drilling, execution of pre-injection, start of the drill and blast procedure etc.

One problem for visual observation of the probe drilling process using modern hydraulic drill jumbos is the automatic jamming prevention system. When the drill bit goes from hard unjointed rock into weak or very fractured material, the feeder pressure will be reduced, and a short retraction may take place. These automatic system reactions to varying rock conditions are not easy to interpret by the observer.

The advantages of percussive probe drilling are the low cost, the speed of execution (which means further reduced cost), no extra equipment being needed, and a fairly high probability of detecting major and serious features ahead of the face.

The disadvantages are the dependency on the observer's experience and the subjective evaluation of what is being observed. It is very difficult to interpret the observations made, apart from high contrast features, so the method is therefore quite basic.

When there are indications that problems of a serious nature will be met within the probing depth, and more exact information is considered essential, a combination including core drilling is often used. This is normally chosen as a next step only after careful evaluation due to the time and cost involved. Core drilling will produce core samples for inspection where the exact location of all features can be logged. In cases where core loss occurs, this is an indication of such weak material that the core has been destroyed (unless careless drilling or worn equipment was the main reason).

Regardless as to whether probe holes are drilled by one or the other method, it is also possible to use borehole radar systems, seismic tomography, electric resistance investigations and similar sophisticated techniques. This is considered beyond the scope of this book and will only be a subject in a very limited number of cases.

6.9.2. Computer supported logging

One example of a commercially available computerized logging system for percussive drilling in rock comes from Atlas Copco in Sweden [6.3]. The system is based on the idea that the penetration rate normally increases in weaker rock and to some extent in jointed rock, while at the same time the torque also increases. The range and pattern of drilling parameter variation gives indications about discontinuity spacing and rock material strength contrasts.

A rock quality model has been developed as PC software, interpreting the measurements made by instrumentation on the hydraulic system of the drill jumbo. The drilling parameters measured and recorded on the PC are:

- Drilled hole length
- Penetration rate
- Axial drill rod thrust
- Torque

The sampling frequency of the drilling parameters can be chosen, and the normal value is generally every 100 mm. When the system is used on a normal drill jumbo, a finer resolution is not ideal, as the borehole deviation and the uncertainty of the exact location of the borehole starting point will be greater than this. Computerized drill jumbos are being used in some projects where the system reads the exact drill jumbo location from a laser beam, and input is made of the exact chainage. If this is combined with stiffer drill rods (larger diameter), then a finer resolution is possible and reasonable.

The graphical model software will filter and process the drill parameter measurements and generate a rock quality scaled color picture.

The entire analysis and presentation on screen takes only a few minutes. The rock quality shown by the colors along the borehole can be compared between holes, rounds or tunnel sections independently of drill depth, thrust etc., which vary extensively between operators. Parallel holes can be combined to generate sectional rock quality maps.

This model is based on a combination of several monitored parameters and is much more accurate and robust than any observation method based on single parameter monitoring. This system also overcomes the observational problem of the automatic drill jamming prevention system. When this system reduces thrust because of weaker material to reduce the risk of jamming, the reduced penetration that results is not misinterpreted as harder rock.
Probe drilling is carried out as part of a decision-making procedure. The information has to be processed and evaluated to decide on the consequences for further activity at the tunnel face. Due to the high cost of time and the often limited possibility of providing highly qualified geological engineers at all tunnel faces at all times, the entire process can become expensive and carry a risk of misinterpretation. The computer system can help substantially in reducing these problems. By using data communication, one geological engineer located almost anywhere may process and evaluate monitoring results from several tunnel faces and return conclusions within minutes. It should be mentioned that the cost of instrumentation, software and a PC to be able to run the system is marginal in comparison to the benefits of the accurate information. The cost of an experienced and qualified visual observer at the face could easily be in the same range, with far less accurate interpretation.

7. HIGH PRESSURE GROUND WATER CONDITIONS

Ground water at high static head (greater than 20 bar), creating high-volume water ingress can cause severe problems in tunnel construction. A summary of experiences from some Norwegian tunneling projects is presented and discussed below. It should be noted that these examples are from the 1970s.

7.1. Basic Problem

In hard rock tunneling through granites and granitic gneisses where overburden is high, only limited parts of the tunnel will intersect highly jointed and sheared areas. Such isolated zones may still produce major concentrated water ingress at high pressure. The characteristics of the problems will vary within a wide range from one project to another. Even smaller and distributed ingress patterns may add up to substantial leakage volumes.

Practical work procedures, economic consequences, construction time and safety are aspects that must be considered and weighed up against each other when considering how to deal with this risk.

7.2. Features that will add to the problem

- High ground water static head (above 20 bar)

Ulla-Førre Project	3 to 15 bar
Kjela Project	20 to 30 bar
Holen Project	20 to 50 bar

- Large water conducting channels intersected frequently and randomly
- Tunneling on a decline, or the access is on a decline or through a shaft
- The tunnel face with its water ingress problems is on a critical path
- Too low pumping capacity for dewatering or poor drainage capacity
- Heavily jointed and sheared rock with clay gouge and fines
- Presence of salt water

7.3. Consequences for the contractor

A normal contract requires the contractor to deal with up to 500 l/min inflow from each tunnel heading. This figure is sometimes as high as 1000 to 1500 l/min. The normal understanding is that the figure expresses the total sum of water inflow over the tunnel section excavated on one face. Such instances of water inflow are normally handled by pumping, especially when they are well distributed. Some local problems may arise, but rarely of a serious nature.

More concentrated ingress directly at the face (a few m^3/min) and at high static head will often create a cost reimbursement situation until the problem zone has been passed. Such an unforeseen situation will cause loss of time, especially when no preparations have been made in advance. On short notice, the contractor will have to provide grouting equipment, grout material, pumps and suitably experienced staff. When such conditions are encountered in a given tunnel, more zones of a similar nature will often be found later. In this case it is a good idea to prepare both contractual and practical solutions for dealing with these situations, thus minimizing the problems.

7.4. Consequences for the owner

The owner's concerns are mostly focused on project progress and staying within budget. The problem is that the economical consequences of delay may be far more serious than the contractor's execution cost. This is often the case if the face is on the critical path of a major project. A rough sample calculation may illustrate the point (cost in USD):

Specialist grouting contractor with equipment	5 000 per day
Face standstill cost, main contractor	24 000 per day
Additional interest cost (Hydro power project 200 Mio. total)	75 000 per day
Grouting material, packers, consumables	10 000 per day
Total	**114 000 per day**

If the above figures are related to a hydropower scheme, the lost revenue from delayed electricity production must also be added. The cost of possible damage to the surroundings is not included.

To keep a complete set of grouting equipment on standby at the job site, the rental cost could amount to USD 500 per day. In addition, there would be some cost for storing grouting material on site (capital cost and potential cost of waste if shelf life runs out). With this rental cost, material cost and the above cost of delay, one day saved project time would cover almost one year of standby cost. The water problem risk would have to be very small not to make it worth paying this premium.

7.5. Methods for handling water ingress

There are two ways to handle water ingress problems in tunnels:

1. Pumping the water out or letting it drain (if possible)
2. Injection to stop the ingress

These methods could be regarded as alternatives, but there are good reasons to consider them as supplementary action to be taken.

- There are clear limits to the quantities of water that can be pumped through pipes or that can be handled by gravity drainage systems when reasonable practical and economical limits are applied.
- Such limitations are even more pronounced when considering only the face area. Tunneling on a decline will already experience serious problems at ingress rates of only 1 to 2 m^3/ min.
- Water at a high static head may produce water jets which spray the whole face area, creating very difficult working conditions. At low water temperatures in particular, the situation will become quite unpleasant.
- If inflows have already occurred (through cracks and joints), post-grouting is very difficult, costly and often unsuccessful.
- It is possible to successfully grout almost any kind of water conveying feature, provided that detection and contact has been made through drilled holes (pre-injection). The prerequisite for this is effective probe drilling ahead of the face, which is a relatively simple and inexpensive measure.
- Pre-injection may also be costly and time consuming, particularly if the aim is absolute water tightness. Experience shows that an attempt to seal the last 10% of a potential water flow may cost more than sealing off the first 90%.

Concluding on the above points:

- It may be a high-risk operation to excavate without probe drilling, only relying on the pumps or drainage for de-watering. A drowned tunnel at standstill for weeks or even months may be the consequence.
- A complete sealing of the tunnel by grouting (drip free) may often become too time consuming and costly to be a feasible solution.
- A well planned use of probe drilling, pre-injection where necessary and de-watering by pumping will normally be the optimum solution. The risk of major water inrush can be virtually eliminated.

7.6. Practical procedure in high risk areas

7.6.1. Pumping system

The pump capacity must be chosen based on predicted and actual project conditions. The reserve capacity should be a minimum of 100%. When excavating on a long decline, a stepwise pumping system with buffer tanks and decreasing pumping capacity down the decline has proven very efficient.

7.6.2. Probe Drilling

Probe drilling will improve safety against sudden major water inrush. The safety improves proportionally with the number of appropriately oriented holes, within a practical range of 1 to 10 holes. There are examples of major water inrush not detected because executed probe drilling did not hit the water bearing channels. In high risk areas the minimum should be about 4 holes, and in shafts it would be recommended to drill more.

The length of probe holes can be adapted to the equipment, shift sequences, round length, ground conditions, water head etc. The minimum overlap should be about 5 m. Probe drilling of this type is normally executed by percussive drilling equipment. Diamond core drilling should only be a supplement in special situations, due to time and cost.

7.6.3. Injection

If very large water flows are found at a drilled depth smaller than the planned probe length, only 2 to 3 meters of further drilling is carried out. More holes are then drilled into the same area and injection is carried out. New holes to check the effect of the grouting are made from the same face position if the contact depth is less than about 15 m.

If the first contact and injection is made at a depth greater than 15 m, further excavation as far as 5 to 10 m can be executed to shorten the drilling for control and grouting purposes.

7.6.4. Special Issues

In spite of probe drilling and pre-grouting, there is still the risk of intersecting water when drilling a blasting round. In such a situation, an extra round of grouting may become necessary. If so, care must be exercised to avoid high-pressure water coming too close to the face. This can be achieved by placing packers as deep as possible into the hole, and by limiting the number of holes drilled. In case of poor and jointed rock in the face area, the risk could be high that a local rupture in the face occurs. Grouting will then become difficult or impossible due to the flushing out of pumped material. Experience shows that one week may easily be lost in such a case before being able to establish an anchored concrete face slab, which in turn allows controlled grouting to be executed.

7.7. Practical aspects

At the Holen hydropower project, Øyestøl access (52 m^2), the recorded ground water static head varied between 20 and 50 bar. Such pressure may cause quite dramatic effects in the tunnel. When drilling into water bearing zones, water, sand and fines can punch through the drill rod, making its way back into the drilling machine. Water supply hoses of normal quality which lead to the drilling machine can break. When withdrawing the drill rod, the water jet out of a 51 mm diameter hole can easily reach 25 m back from the face. The water yield from a contact at a 10 m depth would typically be 2 to 3 m^3/min. A measurement made on a 45 mm diameter hole of 4.5 m length would give 4 to 5 m^3/min.

To reach the depth required for safe handling of the problem, drilling has to be executed by the coupling of drill rods. Manual handling of the drill string against 50 bar static head is impossible. The force exerted on a 51 mm diameter drill bit is about one ton. It has accidentally occurred that the drill string was blown out of the hole and landed 15 m away from the face. In a different situation, such a blow-out from the borehole slammed the drill rods into the front of the drill jumbo, producing a very visible dent in the 25 mm steel plate.

When high pressure ground water is expected, the drill jumbo must be equipped with hydraulic clamps for securing the drill string during coupling and de-coupling of the rods. A last resort at extreme pressure without such equipment is to drive the drill jumbo from the face until the drill string is free of the hole. When drilling more holes into the zone, all holes should be drilled to almost full depth. The last one or two rods should then be coupled by moving the drill jumbo to «motor» the rods in and out.

When conditions allow, it is beneficial to drill a number of holes which make contact with the water-bearing zone. The pressure will then normally drop somewhat due to drainage, making it easier to place packers in the holes. The effect of the first step of grouting will normally be better, compared to grouting through very few holes. Normal cements will require about 24 h setting time before control holes can be drilled. Drilling too early may cause a rupture and flushing out of the injected material.

To place packers against static head of 50 bar, adaptations on the drill jumbo need to be made. The drill feeder and drill rod guides must allow handling of the packers using the hydraulic system. It is quite complicated to enter the borehole even with such a solution due to the water spray produced and the resulting lack of visibility.

7.8. Equipment

Grout mixing and pumping equipment must allow a pumping capacity of about 5m^3/h at 30 bar pressure as a typical starting point. The pump must allow a minimum of 50 bar over pressure compared to the ground water static head. Hydraulic pumps should be used as these normally allow independent control of flow and pressure. The equipment package must allow the use of fairly stiff mortars (e.g. no long and narrow suction hoses), and preferably allow particle sizes up to a maximum of 5 mm.

Couplings, hoses, valves and other fittings must be designed for the maximum pressure.

Every type of standard mechanical rubber expanders (packers) will easily become «bottlenecks» in the system, in more than one sense. The rubber expander is frequently damaged during insertion against extreme pressure, or it cannot take the load when closing the valve. The inner pipe used in packers for 51 mm diameter holes is often a ½“ (12 mm) pipe. This pipe is weak and will also limit the maximum particle size in the grout to about 3 to 4 mm. A long pipe of this dimension will furthermore reduce the possible pumping capacity.

A threaded steel pipe with a split set rock bolt in one end has proven to be a good alternative. The net diameter with this solution is about 40 mm. The fixing of a standpipe by quick setting mortar, combined with drilling through the pipe, is sometimes a good and necessary solution (refer to Figure 6.14).

7.9. Examples

Some key facts are given below from executed tunneling where severe ground water problems were encountered.

7.9.1. Kjela hydropower project

The access tunnel Turvelid, with excavation in direction of lake Bordalsvann, is located close to the European E68 motorway from Edland to Haukeliseter. The access tunnel was excavated on a decline, while the water tunnel towards Bordalsvann had a slight incline with frequent ground water inflows. At 1800 m from the access tunnel, the worst water ingress situation occurred.

Ground water at 23 bar static head was encountered in probe holes 6 to 7 m ahead of the face. Grouting was carried out over 6 shifts and was prematurely stopped due to time and cost concerns. The next blasting round struck a water inrush of 15 m^3/min.

Steel pipes (4 units of 102 mm diameter) were placed on the tunnel invert from the inrush point outwards. The last 7 m of the tunnel was then solidly filled by concrete (while the water was drained through the pipes). Additionally, 2 m of concrete plug was laid later to be able to seal the inrush properly. Contact grouting around the concrete plug finally brought the remaining inflow with closed steel pipe valves down to 500 l/min. When the situation was under control and a by pass tunnel had been excavated, about 6 months had been lost.

7.9.2. Ulla Førre hydropower project

The Flottene access tunnel was excavated on a decline, and the water tunnel in the direction of Førrejuvet had some ground water problems. In a similar situation as the one described above, the owner wanted to blast the next round in spite of a likely ground water yield after blasting being higher than the installed pumping capacity. The blasting was not declined by the contractor. The owner then instructed the contractor to provide increased electricity supply, extra transformers and more pumps and pipes to handle the expected water ingress. Meanwhile, the contractor was able to start injection, and 30 tons of cement were placed. A successful excavation through the zone was subsequently finalized before the added pump installations could be made operational, as instructed by the owner.

The Osane access tunnel and water tunnel (75 m^2 cross section) were both excavated on a decline. Frequent ground water problems were encountered. A summary after 2300 m of tunnel excavation showed the following:

- Total quantity of cement injected was 1100 tons
- Added cost due to ground water problems was USD 10 Mio.
- A maximum of 175 tons were injected from one face position
- The typical ground water static head of 15 bar corresponded well with the overburden and GW table

7.9.3. Holen hydropower project

The access tunnel Øyestol was excavated on a decline, and in the water tunnel the ground water static head varied between 20 and 50 bar. Typical problems encountered during injection works were bent and damaged packers, sand and water punching into the drilling machines and backflow of cement through cracks in the face. In two cases failures occurred behind the face in pre-grouted areas which were passed by the face.

Grout failure and water inrush behind the face under such high static head is extremely difficult, but not impossible to stop. In the above mentioned cases, the open joints filled by grout material were typically 100 mm wide, with substantial local variation. The blowouts occurred at wide spots where the depth of cement filling was insufficient to sustain the water pressure.

Steel pipes with a diameter of 75 to 150 mm were driven into as many of the inrush channels as possible. Additionally, relief holes were drilled from the sound side rock into the water bearing zone. When sufficient pressure relief was established, valves were fitted to the steel pipes and the pipes were anchored to the rock using rock bolts. The whole pipe and valve assembly was then encased in reinforced sprayed concrete, also anchored into the rock. After necessary hardening, packers were used to close the relief holes, and the valves on the steel pipes were closed. This exposed serious leakages around the pipes and the concrete plug, which had to be sealed by grouting and additional sprayed concrete while all valves and packers were open. The start of injection required the use of saw cuttings, concrete with particles up to 10 mm in size, and stiff mortar with anti-washout admixture. As soon as the backflow was under control, a stiff cement mortar with a low w/c-ratio of about 0.5 was used for the feature injection.

A prerequisite for the successful sealing of such blow outs was stiff mortar, a low w/c-ratio, injection pump pressure capacity of 50 bar above GW head, sufficient quantity injected, and at least 24 h hardening time before any disturbance could be allowed (such as removing packers or pipes and valves).

7.10. Summary of lessons learned

High ground water static head, high ground water yield, excavation on a decline and other possible problem enhancing features require the following set of measures to be applied to reduce problems:

- Probe drilling ahead of the face must be executed on a routine basis. The amount of pre-grouting must be balanced against the cost of pump and pipe installation and operation of the system. Pre-grouting and dewatering by pumping must both be carried out (these measures should not be viewed as exclusive alternatives, but must be used in combination with each other).
- The reserve pump capacity must be at least 100% more than the maximum expected water inrush.
- A back-up diesel generator is often required to ensure the supply of electricity to the dewatering pump system.
- It is a requirement that the grouting equipment has sufficient capacity regarding flow and pressure, and the ability to pump particle size up to 5 mm.
- Post-grouting is difficult and time consuming and may become impossible.
- Pre-grouting, on the other hand, is simple and efficient, provided that a tight face area is maintained. A 5 m buffer zone is recommended in sound rock. In weak and poor rock a larger zone is required.
- High static head requires care and special measures. One must not allow high-pressure water to come too close to the face, particularly in poor ground. 10 m of buffering zone will not help if the packers are placed only 2 m into the borehole.
- Cement should not be the main basis for payment as it is a relatively small and highly variable cost factor. Even at a high consumption rate of cement, the cost of cement is only about 5% of the total contractor's cost.
- Depending on conditions, the added cost of pumping, probe drilling and pre-grouting may be in the range of 50 to 100% of the standard excavation cost.
- Finally, the face area must be kept watertight, and if in doubt, the next round should not be blasted.

8. MAXIMUM PUMPING PRESSURE

8.1. Introduction

The maximum allowed injection pressure for a given set of conditions or project frequently causes heated discussions arising from two different ways of looking at the subject:

1. The low-pressure approach where the focus is on not creating damage in the rock structure around the tunnel or anywhere in the surroundings of the project.
2. The high-pressure approach where the focus is on getting the job done efficiently both regarding time, economy and quality of the result.

It may be seen as a little unfair to present the two views in this way, as it might suggest that the first approach supporters have no interest in efficiency and result. The second approach may well be carried out without causing «damage», providing that some simple safeguards are employed.

The author started working with rock injection in 1971. At the time, the low-pressure approach was the dominating one. Over the years and through many projects, there has been a gradual shift to the high-pressure approach as experienced by the author. It is a very strong argument in favor of the second approach, as this development has gradually produced substantially improved grouting results without causing any damage to the projects or the surroundings. This being a documented fact of practical project execution, we may proceed to try and explain why this is the case, and will avoid discussing whether this is the right approach, or question if it is even possible.

8.2. Basic background considerations

It may be useful to clarify one point first. If there are no stability problems and no water ingress problems nobody would think about executing any pre-injection. We know that it is still possible to pump water at such high pressure that hydraulic fracturing (creating cracks in solid virgin rock) could occur. This could be classified as damage, as the quality of the rock would be reduced. Furthermore, if the medium pumped is cementitious grout rather than water, the strength of the cured grout would normally still be less than the strength of the split rock volume, resulting in a reduction of quality.

However, grouting in real situations is executed to control ground water flow and/or to improve stability of the rock formation before excavating into it. Both of these potential problems exist because of cracks, joints, channels, low friction joint material, clay, crushed shear zone material etc. and sometimes rather high hydrostatic ground water head (e.g. > 20 bar). It should be quite easy to agree that the purpose of pre-injection in such cases can only be satisfied if the grout can be placed into those openings and discontinuities by the use of pumping pressure.

The maximum pressure specified for pumping of the grout is normally given as a net value, in addition to the local hydrostatic head. However, when starting injection on a hole, there has normally been a lot of drainage from the drilling process before any packers can be installed, so the practical GW head will usually be substantially lower than the original head.

8.3. The low-pressure approach

The low-pressure approach was normally executed with the use of cement and Bentonite and very high w/c-ratio (typically >3.0). This required the grout to refusal technique to counteract the negative effects of the unstable and bleeding grout. To summarize the effects of this method, the following should be noted:

- Many boreholes would take very little or no grout even though water pressure testing showed leakage, or yielding to ingress water. Depending on geological conditions, there would be little or no grout take e.g. at a pressure of less than 15 bar, while later experience shows that most holes would probably allow grouting above this pressure level. Too low maximum pressure would result in many holes being lost, as they could not contribute to the targeted result by allowing grout take to the surrounding ground.

- Holes that would take grout ended up taking far too much because pumping had to continue until the maximum pressure could be reached. This low maximum pressure would not be very efficient at squeezing surplus water out of the grout. In effect, the grout that was placed would bleed and cause incomplete filling of the openings, allowing residual leakage. The strength parameters of such grout would also be low, contributing little to improved stability.
- The low-pressure approach has historically NOT prevented damages from occurring, (and this method could be the very reason for the concern about the effects of using higher pressure). One reason for this is the grout to refusal technique. The lifting force exerted by pressurized grout is a result of the area under pressure multiplied by the average grout pressure. With a low-viscosity water-rich grout, the pressure loss from the pump to the grouting front is relatively low, especially at the end of the procedure where the maximum pressure must be kept for typically 5 to 10 minutes at a very low or no flow rate. With no limit on grout quantity, the area could be very large and the lifting of overburden could occur even if the maximum allowed pumping pressure was considered to be low.

8.4. The high-pressure approach

If a low-pressure approach means 10 to 20 bar maximum, what will happen if this maximum is raised to 5 times more? Understandably, the first reaction is that it will cause damage and would therefore be unacceptable. However, as mentioned already, reality shows this NOT to be the case and there are some very good reasons to explain this. The high-pressure approach is typically combined with stable, non-bleeding grout and individual boreholes are stopped either on specified maximum pressure or a maximum quantity, depending on whichever is reached first. To summarize the effects of this methodology, the following must be noted:

- The holes will normally start taking grout at circa 30 to 50 bar. Under the ground conditions described above, the existing discontinuities around the borehole are dilated (opened, deformed, forced to widen locally), creating a connection to existing open joints and channels in the ground, and then grout can start flowing. The pressure will rapidly peak reaching 30 to 50 bar or more, and then suddenly drop to a much lower level. Most of the pumping can then be carried out at this lower level. «Damage» has been done in the sense that some existing weakness has been forced to open locally, but this happens in the immediate vicinity of the borehole (probably less than 1–2 meter scale) with no effect anywhere else.

- It may be said that the existing weakness that was forced to open up is now even more of a weakness than before. This cannot necessarily be agreed with because the result depends on the quality of the grout injected. When using a stable grout without Bentonite, the compressive strength of the cured grout will be in the range of 5 to 10 MPa, the grout is non-bleeding and the filled discontinuity will show improved strength parameters and lowered permeability compared to the virgin situation.
- If the grout take in a borehole starts at a high pressure level (circa 40 bar) and it continues to rise during pumping (no sudden pressure drop and probably no local dilation of any opening), then all of the pumping on this hole may happen at high pressure and finish at the specified maximum. The maximum pressure will then be reached long before the specified maximum grout quantity per hole (e.g. maximum 3000 kg of cement). In other words, the lifting force is limited by the area under pressure because of the limited grout quantity, and no damage is done.
- If both the specified maximum grout quantity per hole and the maximum pressure allowed are reached more or less at the same time, this will represent the theoretical maximum lifting force (and risk of «damage»). The typical maximum grout quantity specified for this approach is still just a fraction of the necessary volume in grout to refusal, so the force is still quite limited. As an example, the author has personal experience from pumping 150 tons of cement on a single hole (grout to refusal) and has found grout migration coming to the ground surface 600 m away from the point of injection.

If particular care is necessary because of pressure due to nearby structures, low overburden (less than 20 meters) or any other special circumstances, then the GIN principle or a modified version of it can be employed. In its simplest form, the maximum allowed pressure can be linked to the quantity pumped. If for example the general stop criteria are 3000 kg cement or maximum 60 bar pressure, then an additional requirement could state that the maximum pressure above 1500 kg cement pumped would be 45 bar. This way it is easy to limit the maximum theoretical lifting force without losing the advantage of high pressure to start the operation and provide penetration.

8.5. Summing up

When ground conditions and/or the hydrological conditions are such that pre-injection is necessary, then the targeted result can only be reached if sufficient grout volume can be placed in the right locations. These locations are always found where the ground is already jointed, broken, crushed, weathered and otherwise of reduced quality. Good, stable cement grout, will improve conditions compared to before, even when placed partly through forced entry.

In principle, the maximum allowed pressure should be the pressure necessary to place the necessary grout volume where required. To place the grout and to achieve the targeted benefits of the drilled holes, some local deformations in the ground immediately around the borehole may be required (as caused by high pressure). This cannot be termed damage as no negative effects can be identified.

8.6. The theory behind high pressure grouting

Dr. Nick Barton presented a paper with the above title, printed in Tunnels & Tunneling International in September and October 2004 (parts 1 and 2) [1.4].

The paper clarifies on a theoretical basis the reasons why high pressure (50 to 100 bar) does not cause «damage», and why it also substantially improves injection results. It is strongly recommended to study this paper carefully if further substantiation of the high-pressure approach is needed. Some of the most important aspects of the paper can be summarized as follows:

- The use of 10 bar water pressure testing will have a negligible effect on the existing joint apertures under most conditions. This is one reason why such testing normally shows no correlation with grout takes, and is frequently not executed as a routine test in tunneling.

- The physical joint apertures E are larger than the theoretical hydraulic aperture E. This is easy to understand when considering that natural joints (E) have roughness and rock/rock contacts across the joint plane, but they still convey the same water flow as the smooth and completely separated walls at a distance of e apart.
- The physical joint apertures may open up between 10 and 50 micron close to the borehole, when 50 to 100 bar injection pressure is used. This could represent an increase of E by 20 to 100% and clearly important for the creation of grout permeation by avoiding joint entrance blockages through filtration.
- When using an ultra fine cement with d_{95} = 10 micron, the joint opening would have to be >40 micron for the grout to enter (according to the formula used by Barton). Permeation could then be achieved on joints with original apertures between 20 and 33 micron due to the widening by injection pressure.
- A very positive side effect of high pressure grouting is discussed by Barton. The average ground stability or ground quality as expressed by the Q-value is substantially improved, and this will influence the cost and time of construction in a very positive way.

9. EQUIPMENT FOR CEMENT INJECTION

There are many ways of executing injection with cement, and because of the relatively low cost of the material it is sometimes believed that the equipment side can be improvised with little negative consequence. This is a serious misunderstanding, and the result of such an approach to cement injection would be an overall cost increase, poor efficiency and substantially lower quality of the work done.

There are many specialist manufacturers producing high quality equipment for cement grouting. As they are so numerous, they cannot all be presented here. It is not the purpose of this publication to grade or recommend any particular manufacturer. However, it is strongly recommended to select a complete set of equipment from one of these specialist companies before starting any sort of grouting operation underground. If the requirements call for the use of micro cement it would be a considerable waste of more expensive material not to use modern, custom designed, dedicated equipment.

Companies such as Atlas Copco Craelius, Häny, ChemGrout, Montanbuero and Colcrete are all good options when seeking an equipment manufacturer, but as mentioned, there are many others. The choice has to be made based on local requirements and a detailed evaluation in each case.

9.1. Mixing equipment

The process starts by mixing dry cement powder with water and often other components of the mix, such as chemical admixtures, Bentonite, sand or other materials. The crucial point is to get all the cement particles completely wet.

This may seem a simple task, but trying to do this manually with just a small cement quantity is not that straightforward. Once all of the cement looks wet, not all the individual particles may have actually come into contact with the water. Fresh cement will start to agglomerate with air humidity, and at the time of mixing, a large amount of the «particles» are actually agglomerations of several individual particles. With the focus on cement particle size to achieve penetration, it is obvious that the theoretical particle size will be quite different from the practical size unless the agglomerates are dispersed. This can only be done by high shear dedicated equipment.

Cement mixing equipment for injection works will fall into two main categories:

1. Mixing by agitation
2. Mixing by high shear action

The first method is typically represented by a sort of paddle mixer as illustrated in Figure 9.1. The agitation creates turbulence in the mix, and after some time it will appear to be uniformly wet. The drawback with this method is that it will not fully break up dry lumps and agglomerates consisting of many individual cement particles. The surface tension of water tends to preserve such lumps and this creates grout segregation, blocking of small openings and build-up in bends, valves and other parts of the equipment. The positive effect of a longer mixing time is quite limited and will not solve the problem.

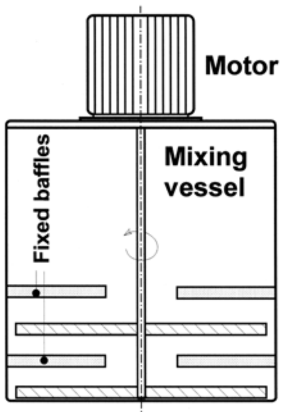
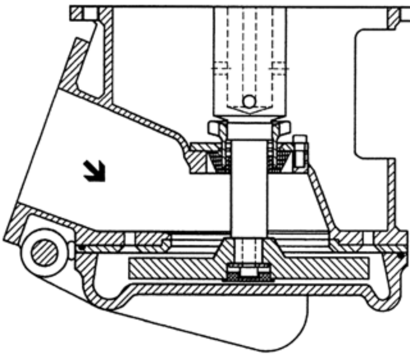


Figure 9.1 Paddle mixer [9.1]

The high shear mixers are normally termed colloidal mixers. These units typically consist of a tank with a high-speed circulation pump. Water and cement is drawn from the bottom of the tank, runs through the high-speed impeller of the pump and returns to the top of the tank. A good colloidal mixer will have an impeller speed of 1500 to 2000 rpm and the shear action is strong enough to break up lumps so as to properly wet individual cement particles.

The high shear action is created either by the tight tolerance between the impeller and its housing, or by intense turbulence (see s 9.2 and 9.3 respectively). The whole tank volume should be fully circulated at a rate of about three times per minute. It should be noted that the principle shown in Figure 9.3 is best suited if there is a need to add sand or other coarse materials to the grout due to the lower wear cost.



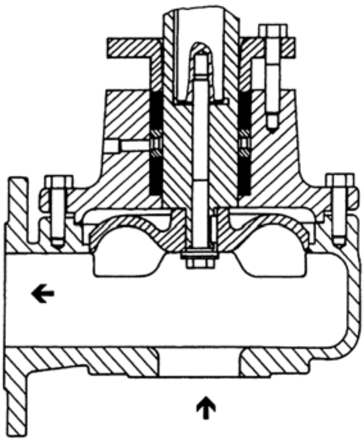


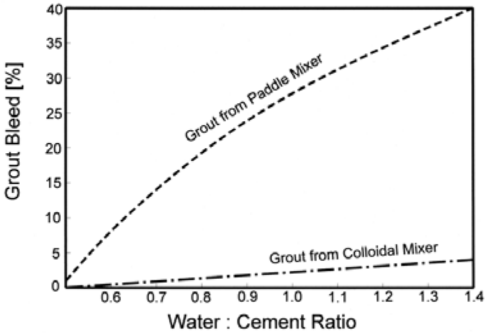
Figure 9.2 Tight tolerance [9.1] and 9.3 Turbulence [9.1]

The difference in mixing efficiency between colloidal mixers and other types is easy to demonstrate by simply comparing the grout behavior of equal mix designs and mixing times in the two types of mixers. Tall glass cylinders filled with grout will demonstrate a substantial difference in bleeding. Pouring grout from a paddle mixer onto a low and wide plate and allowing it to harden, will show distinct layering when breaking up the cement cake. The same test gives a uniform layer using the colloidal mixer. See figure 9.4 for a picture of a standard colloidal mixer.

Figure 9.4 Typical colloidal mixer (Photo Atlas Copco)

Cement injection is mostly carried out against ground water flow. Paddle mixers will create a grout that has a strong tendency to be diluted and washed out by the ground water. With a colloidal mixer the grout is much more stable and will tend to displace the ground water rather than mixing with it. By simply lowering a spoonful of grout into water and turning it upside down (to allow the grout to fall through the water), the difference becomes highlighted. The grout from the paddle mixer will totally dissolve, creating a total cement cloud. One can observe how the other grout falls like a lump, creating much less of a cement cloud.

A. E. Reschke [9.2.] carried out comparative mixing tests using ordinary Portland cement in a Colcrete SD4 colloidal mixer and in a Thiessen Team TC3100 paddle mixer. Mixing time in the colloidal mixer was 1 minute, and 15 minutes in the paddle mixer. The substantial differences in grout quality are well demonstrated by the figures 9.4.1 and 9.4.2. The compressive strength of a paddle mixed grout will also suffer.



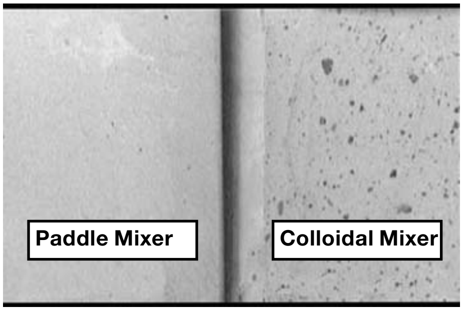


Figure 9.4.1 Cement grout bleed and 9.4.2 Mix homogeneity

One must be aware that the high energy used in a colloidal mixer will raise the temperature of the grout volume. This is not a problem in normal operations carried out as specified, but if a mixing time is used that is too long, the batch may be unusable and could set in the equipment. Micro cement of the fast setting type could be very sensitive to this effect, so the mixing procedure must be well controlled. One batch of grout is generally created in about three minutes. The circulation pump is then used to send the prepared batch to an agitated holding tank, from which the injection pump draws grout. Therefore, even though the mixing is done in batches, the pump may still operate continuously.

Colloidal mixers are made in a variety of sizes, and there must be a balance between the maximum pump output and the maximum capacity of the mixer. The equipment manufacturers will normally offer well balanced equipment sets to suit the needs of a customer. It is recommended to only use weight batching of the grout components as it is much more accurate. Liquid components may of course be added volumetrically, provided that reliable measuring devices are employed.

9.2. Grout pumps

To be able to execute well controlled high pressure grouting in rock it is necessary to have a suitable injection pump. Even though progressive cavity pumps (see Figure 9.4.3) have been used for decades in numerous rock grouting projects (primarily dam foundations and other above ground projects) it is clear that this type of pump is not suitable for most underground tasks today. There are many reasons for this, but the most important is the limited maximum grout pressure, the high wear cost on rotor and stator, and the impractical pressure control system provided by a return line back to the hopper through a flow control valve.

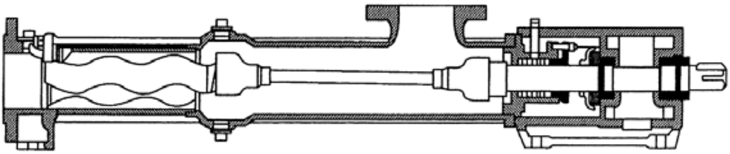


Figure 9.4.3 Progressive cavity pump (mono-pump)

Today's preference in underground grouting projects is the piston plunger pump with its hydraulic drive system. Such pumps will normally work on a single grouting line. This pumping system requires and allows independent grout pressure and grout flow rate control without any valves or mechanical control parts coming into contact with the grout. For wear and reliability reasons this is especially important at high pressures. The operating reliability and control accuracy is very good with this methodology. Furthermore, the plunger pumps have the advantage of low wear even with abrasive grouts, and they operate reliably at a very low output rate. High pressure may be maintained over time at marginal or no output.

There is full agreement that tight pressure control is required to avoid exceeding the specified allowed maximum pressure. Pressure peaks above the set level at the start of a piston stroke (due to inertia of the grout column) is unfavorable, and it is not a characteristic of modern equipment. In critical cases, pressure above the limit may cause damage to nearby structures or cause unwanted fracturing of the ground.

Regarding the effects of the pressure pulsation which is normal for piston and plunger pumps, there is a level of disagreement in the industry (see 9.5). Some say that a constant pressure and flow is best, whilst others say that the pressure drop between pump strokes is actually an advantage. Practical experience supports the idea that pressure drops actually improve grout penetration [9.1]. The reason for this is the rearrangement of particles that are about to bridge and block a narrow joint (causing pressure filtration and full blockage) when pressure suddenly drops. When the pressure increases at the next stroke, the same particles may again move forward, but this time without bridging (see Figure 9.5).

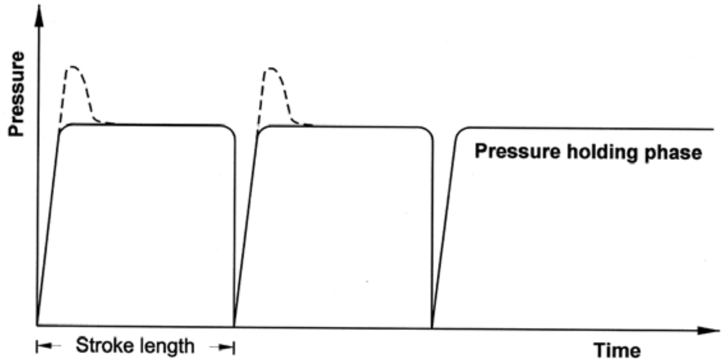


Figure 9.5 Pressure pulsation by piston plunger pumps [9.1]

9.3. Complete equipment systems

Today, most manufacturers of grouting equipment offer complete systems with all elements included (mixer, agitator and pump), frequently including a PLC control of batching with the mix design ratios pre-stored in its memory. For such systems to operate properly there must also be an integrated weighing component and accurate measuring devices for water and admixtures.

The layout of complete systems can vary to quite an extent, and their size may range from small compact units to be put on a small truck or trailer, to larger units that will need a heavy dump truck chassis or similar. The larger units may have hydraulic working platforms to allow access to packer placements in the roof of the tunnel, which can be more than 10 m high in larger highway tunnels.

One example of an assembled complete system is shown in Figure 9.6.

Figure 9.6 Complete system (Photo Atlas Copco)

A high-output equipment system for pre-excavation grouting in larger tunnels has been assembled by general contractor AF Spesialprosjekt AS/SRG of Oslo, Norway (see Figure 9.7). The equipment unit contains 2 colloidal mixers, 4 agitator vessels and 4 hydraulic pumps, each pump capable of delivering 60 l/min at 100 bar grout pressure. The whole system has been built into a container, which is carried on a normal heavy-duty road truck.

Figure 9.7 Container mounted multi-pump grouting system (AF Spesialprosjekt AS/SRG)

9.4. Recording of grouting data

The traditional way to record injection data has been manual recording of the main injection parameters by reading data from instruments and writing it into pre-printed forms at defined intervals. Grouting pressure, cement quantity, type of mix design, choice of hole, and general data such as location, date and time are noted this way. This part of the work may require an extra person just for record keeping, especially if the procedures are complicated and many different parameters have to be accurately recorded.

As time is money at the tunnel face, often more than one borehole gets injected at a time. The manual recording task then quickly becomes impossible.

Today there are a number of alternatives available to improve data recording and to reduce the workload for the operators. The simplest device is a pressure transducer and an inductive flow meter coupled into the grouting line, transferring the data to a chart recorder. The printed data sheet can be collected when the hole is finished, and the unit may be reset so that the next hole can be started. Grout quantity may alternatively be recorded based on pump stroke impulse counting with quite reasonable accuracy (see Figure 9.8).

Figure 9.8 Example of a simple grout recorder (Photo Atlas Copco)

Regardless of how the system is set up and how many automatic recording devices are used, it is important that good visual control of injection pressure is available at all times. A good manometer with a simple and clear scale must be installed in a place where it is easily observed (see Figure 9.9). It should also be noted that the measuring range of the installed manometer must reflect the practical grouting pressure range on site. Sometimes manometers are installed for 0 – 100 bar, while the pressure range allowed may be only 0 – 10 bar. In such a case, the resolution will be quite unsatisfactory.

Figure 9.9 A good and easy to read manometer is important

More advanced versions will send the data to an electronic data logger, but this is just a different way of recording the same data, and is a very practical way to do it. When using data processing (a PC), this adds the opportunity to actively control the process from the PC. Control parameters such as maximum allowed injection pressure, maximum and minimum flow rate and maximum quantity of grout per injected hole can be entered in to the PC. It will then record the process automatically, but also stop the pump when any of the stop criteria have been reached. When injecting on several holes simultaneously (with one pump per line) this equipment is a great help in keeping things under control and receiving accurate recordings, without the need for more staff. At a tunnel face with extensive grouting it will be cost-efficient and increase work quality and effectiveness.

10. METHOD STATEMENT FOR PRE-INJECTION IN ROCK

This Method Statement is written specifically for the use of RHEOCEM micro cement or for a micro cement with similar properties. The most important features that must be satisfied for this Method Statement to be applicable are:

- Stable grout with less than 5 % bleeding (normally zero), thixotropic behavior, Marsh cone viscosity of less than 35 seconds, quick setting grout and good pressure stability (low filtration coefficient).

Soil injection is not considered here. This Method Statement is primarily intended for competent rocks from medium hard to hard, including the normal frequencies of weak zones and particularly jointed and crushed zones. Such tunneling is typically carried out by drill and blast, and this is the excavation method considered in this document. The same principles will also be applicable in a hard rock TBM tunnel, and this Method Statement can be developed and modified to also cover this excavation method, however it is not included here.

In a practical case with very strict water ingress limitations it would be beneficial to combine the use of RHEOCEM micro cement and the colloidal silica MEYCO MP 320. For control of backflow problems and in post-grouting situations, the range of one and two component PU products of the MEYCO MP 355 series could also be used as supplement.

10.1. Drilling

10.1.1. General

Drilling of probe holes and grouting holes is done with the multi-boom drilling jumbo which is primarily there for the blast hole drilling. A typical drill bit diameter is 51 mm or 64 mm, with rods and couplers which fit the drill bit selected. During drilling, the penetration rate, occurrence of weakness zones, water (or loss of flushing water) and other selected parameters are observed and recorded in a prepared format by the drilling supervisor or operator.

Together with the measured water yield from the drilled holes, this record forms the basis for the action to be taken, e.g. choice between injection or no injection, and if injection is chosen, how many additional holes, what length etc. See Figure 10.1, the decision flow chart at the end of this chapter.

10.1.2. Flushing of boreholes for injection

The first requirement is good water flushing during the drilling of the hole. The water pressure used should be at the maximum specified by the drilling equipment manufacturer, which is ensured by a special pressure booster pump on the drilling jumbo.

Further cleaning of the injection holes must be described as either a procedure combining water and compressed air, or by high pressure water cleaning as described in chapter 6 and Figure 6.8.

Flushing by water and compressed air should be done using a stiff plastic hose using water at 10 bar pressure, combined with some compressed air. Push the hose to the bottom of the hole, open for water and air, and withdraw the hose while flushing is turned on. If there are zones in the borehole that may collapse if soaked in water, or will be excavated by the flushing jet, or if the water yield from the hole is more than 10 l / min, the flushing may be omitted.

Flushing of boreholes for grouting should be done as specified as a routine matter and any necessary deviations should be decided on and recorded by the supervisor, based on the borehole records.

10.1.3. Length of boreholes

Probe holes are normally less than 30 m long. The length specified may be influenced by the chosen borehole diameter, as the deviation is substantially larger for the 51 mm equipment than for the 64 mm equipment. Normally, a balance between drilling accuracy and the risk of getting stuck is aimed at for injection efficiency and efficient tunneling progress. If four to five rounds can be blasted between probe drilling (and possible injection rounds), this is the standard choice.

10.1.4. Number of holes, hole direction

Generally, holes are started from the tunnel face very close to the tunnel contour, using a look-out angle of between 5º and 8º, creating a cone pattern with the top cut off (the face being the cut-off plane). There are situations with very dominating joint orientation that may call for an adapted preferential borehole direction, but this is usually not necessary or beneficial.

Probe holes are drilled to reduce the risk of unforeseen water inrush and to detect areas where pre-injection must be carried out to meet ingress limitations. The probability of problem detection increases proportionally with the number of holes drilled to a certain maximum number. Decision on the number of holes must be based on the size of the tunnel, risk involved (inside and outside the tunnel) and the required tightness of the tunnel. This issue shall be covered by the Technical Specification for the project.

When pre-injection has been decided on, the initial number of holes for a first stage injection will typically produce a borehole spacing at the face of 1.5 to 3.0 m. Subsequent stages (if necessary) will be drilled using the split spacing principle. Concerning probe holes, the spacing of the first stage injection holes must be specified.

10.1.5. Placing of packers

The packer is normally placed near the borehole opening and the hole is injected over its entire length in one single step. The packer placement depth is typically 1.5 m. However, allowance must be made for a number of different possible situations that may require a different packer placement.

High ground water pressure and very poor rock may provoke a face failure if the packer sits too close to the tunnel face. It is not possible to give a general rule for this situation, but it may be necessary to place the packer 5 m into the hole. If a channel causes water and grout backflow to the face, the packer must be placed at a depth larger than the depth of the intersection between the borehole and this channel. Sometimes the borehole is locally disturbed by weak rock material, local wedge fallout and similar occurrences, causing the packer to slide or to leak. Placing it deeper will normally solve the problem. In principle, there should be an overlap of tight rock (a buffer either from the sound rock or grouted rock from the previous injection round) of circa 5.0 m in front of the face. The packer placement should be in this zone unless there are reasons to do it differently.

10.2. Injection

10.2.1. General

The decision criteria for pre-injection to be undertaken must be specified. This is often based on measured water in-leakage from the probe holes and can be a given number of l/min from a single hole or a maximum sum leakage from all the probe holes, whichever happens first. Depending on the target maximum water ingress into the tunnel, the injection could be initiated if a single hole yields more than 4 l/min, or if any combination of probe holes yields a total of more than 15 l/min. The balance between these criteria and the target tunnel tightness must be based on experience and local rock conditions, with the option of feedback from results during operation.

RHEOCEM 650, 800 and 900 should be mixed with a w/c-ratio of 1.0 using RHEOBUILD 2000 PF at a dosage of 1.5% of the cement weight. If there are reasons for deviation from the above given parameters or material choice, this must be made by the injection supervisor, preferably in consultation with the material supplier.

10.2.2. Mixing procedure

i. The cement mixer must be a state of the art colloidal mixer with an impeller speed of no less than 1500 rpm. The mixer must also be kept in good maintenance to work efficiently with micro cement.
ii. Add all the water for one batch into the mixer.
iii. Add the corresponding required quantity of cement.
iv. Add the RHEOBUILD® water reducing and dispersing admixture.
v. Mix for 2 minutes. Be careful not to exceed the mixing time, as the intensive high shear mixing will generate heat and increase the temperature of the mix. If the temperature becomes too high, the open time of the batch could be substantially shortened, and in hot climates this could particularly cause practical problems. Likewise, do not cut the mixing time short, otherwise the flocculated clusters will not be broken up by the mixer. Sufficient mixing time and the use of RHEOBUILD 2000 PF is required in order to break up the clusters.
vi. Immediately transfer the batch to the agitated holding tank and keep the grout in slow agitation at all times. Monitor the quantity of grout in the agitator and never start mixing a new batch if the agitator holds a lot of material and the grout pump is delivering at a slow rate and at high pressure. The batches in the agitator should always be kept as fresh as possible.

10.2.3. Use of accelerator in the grout

There are situations when unexpected backflow can occur through the face or even further back in the tunnel. Sometimes indications show that a borehole is in contact with extremely large channels with a lot of high pressure water. In both situations it can be beneficial to accelerate the cement setting and hardening: In the first case, by stopping the backflow and allowing further injection of the ground without loss of material, and in the second case, by stopping unnecessary spread of grout at a reasonable distance from the tunnel. Early strength may also become very important to withstand the forces from pressurized water.

As example the MEYCO SA 162 is normally used as a sprayed concrete accelerator, but it works very well with RHEOCEM grout in injection works. One advantage is that there is no evident flocculation or thickening at the time of addition, and the reaction only influences the grout after a certain time. The dosage of MEYCO SA accelerators can be adjusted to give the effect needed (should only be added through a non-return valve at the packer).

Before using MEYCO SA accelerators, site tests have to be executed to determine the open time, setting time and hardening time with the equipment, type of RHEOCEM and the w/c-ratio used on site, site temperature etc. This is very important to avoid unexpected early setting and risk of premature blocking of the holes.

Addition at the packer:

i. An addition at the packer has to be made by a separate pump and delivery hose, connected to a non-return valve coupled into the grout line at the packer head. This non-return valve is described in chapter 4 and is illustrated in Figure 4.4.

The pump for MEYCO SA accelerators can be a diaphragm pump or a hydrostatic pump, giving maximum pressure higher than the grout injection pressure. The output is adjustable, and because of the high pressure capacity, it will not be influenced by any variation in pressure in the grout line. The accelerator pump has to be linked to the grout pump in a way that ensures the same dosage of accelerator even if the output and the pressure change on the grout pump.
Normally the output of the accelerator pump is controlled by frequency or by using the hydraulic system. Based on pre-testing of the dosage and calibration of the pump, the dosage of MEYCO SA accelerators can be started at any time needed and can be increased in steps until the targeted effect has been achieved. At any stage, the dosage of the accelerator can be stopped and injection may continue with normal grout, keeping the injection connection open.

10.2.4. Injection pressure

As is always the case, the maximum injection pressure has to be evaluated on a running basis and must be checked against local conditions in the tunnel. Very poor rock conditions in the face area, high hydrostatic water head and existing backflow will be indicators that maximum pressure must be limited, even if the rock cover spans hundreds of meters.

In pre-injection the maximum allowed pressure should be used from the beginning of injection (if the pump can deliver sufficient output to reach this pressure) until:

i. No more grout is accepted by the ground at the maximum allowed pumping pressure
ii. The maximum specified grout quantity for the hole has been reached, regardless of pressure used

The choice should be made depending on whichever event happens first. With this approach the quantity of grout that can be placed will be pumped in the shortest possible time period and by working at the highest possible/allowed pressure from the start of the process. This will provide a maximum penetration of fine cracks and joints.

The permitted maximum grouting pressure should be at least 50 bar above the static ground water head, unless special reasons have been identified that require pressure to be limited to a lower level. In pressure sensitive situations it must also be recognized that the danger of damage being caused by lifting, splitting or other deformations is also linked to the product of pressure and grout quantity, and not to pressure alone. High pressure exerted only on the borehole walls (quantity just sufficient to fill the borehole) cannot possibly cause any «damage» anywhere else than the first dm or so around the borehole itself. This potential «dam-age» zone would of course increase both with increased pressure and grout volume pumped.

10.2.5. Injection procedure

i. Start the injection of the lowest hole in the face and work upwards. Alternatively, the holes with the largest water inflow to the tunnel should be grouted first.
ii. A hole is finished when the maximum allowed pump pressure gives less than 2 l/min of grout flow during a 2 minute time period, or when the specified maximum grout quantity per hole has been injected.

If backflow of grout and water into the tunnel is detected, this should be minimized by reducing the pump output, and accelerator MEYCO®

SA accelerator should be used to create a blockage of the backflow. A decision must be made as to which method to use (addition to the mixer or by separate pump).

iii. If during the injection process two or more holes become interconnected as demonstrated by grout backflow through the placed packers, close the packer valves in the connected holes and continue grouting on the current hole. The maximum amount before stopping should be multiplied by the number of connected holes. If the maximum pressure is reached before the maximum quantity, the connected holes should be injected too if they take any grout.

10.2.6. Injection records

Records of the injection data must be taken routinely. Part of this may be done by computerized recording if the system is suitably equipped. Otherwise, well prepared forms must be available for use in the tunnel during work progress. The person responsible for record keeping must also be clearly defined.

The following information is the minimum that must be recorded:

i. General data such as tunnel location, date, time and shift, person who does the recording, identification and location of holes, measured water flow from the holes.
ii. Per hole: Packer placement location, length of hole, grout mix design, start and end pressure, start and end time, flow rate development, total grout quantity, start and end pressure, any backflow and/or connections to other holes.

10.3. Grout setting and time until next activity

RHEOCEM is specifically developed to behave as a thixotropic grout and to give initial and final setting a short time after the end of injection. Its purpose is to allow work to proceed without breaks. At moderate ground water head (circa less than 15 bar) and if water bearing channels are limited in size (e.g. maximum opening less than 10 mm), this should be possible without any risk.

When the pressure increases, the risk of grout material failure and wash out will increase rapidly, especially if the channel dimensions increase at the same time. It is not possible to give general rules on how to evaluate this, other than pointing out that consequences of failure, time allowed for grout setting, and the water pressure and channel size in the ground have to be considered.

If an accelerator has been used to shorten setting time, one must be aware that this will be a good help for the accelerated grout, but in many cases only part of the grout injected has been accelerated. Caution must be taken if the consequences of failure are serious.

If the next planned activity is the drilling of boreholes for control of injection result or for a next round of grout holes, drilling should always be started in the area where the previous injection was first completed (giving the maximum undisturbed setting time).

10.4. Drilling of control holes

The efficiency of a stage of injection must be controlled by new boreholes. These holes will be evaluated using the same decision criteria as used for the probe holes in regard to injection or no injection. Control holes should be drilled on both sides of all holes that yielded water flow above the limit for injection. If the project requires the use of colloidal silica, the decision criteria for a second stage (or subsequent stages) must reveal when to use such grout.

Holes that are tight should be filled by stable cement grout. If no injection is necessary this should take place for all holes. It can be done with rock bolting mortar if preferred, to avoid starting up and cleaning all the injection equipment only to backfill the holes.

10.5. Measurement of water ingress in excavated parts of the tunnel

Control of achieved tightness in the tunnel behind the face is the only way to confirm the result of carried out injection. After a certain length of tunnel excavation, the average water ingress over this length of tunnel must be checked. By installing sealed dams in the tunnel invert with V-shaped overflow, the ingress over this defined tunnel section can be measured. To get accurate readings, it is normally necessary to take measurements at the end of a weekend to avoid disturbance from other activities that use water in the tunnel.

If the required maximum rate of ingress has been exceeded, post-grouting of the remaining leakage spots must be carried out, starting with the largest ones. Furthermore, an evaluation of the pre-grouting procedure and criteria will demonstrate if any adjustments are needed so that the requirements can be met for coming tunnel sections.

10.6. Decision-making flowchart, example criteria (Figure 10.1)

Step I: Probing ahead. Standard number of holes is two in clock positions 12 and 6. In high risk areas, use four holes in positions 6, 9, 12, and 3. Maximum drilling length per hole is 30 m. Use percussive drilling with water flushing. Recommended drill bit: 51 mm diameter. Start holes at tunnel contour, angle out 5° to 8°. Overlap with end of the last drilling is minimum 5 meters.

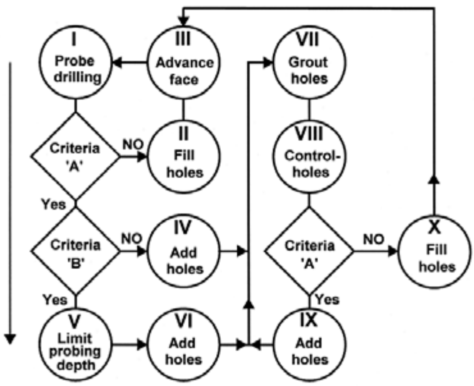


Figure 10.1 Flowchart of decisions regarding probe drilling and injection

Recordings to make during probe hole drilling:

- indications of any weak zones (depth and length), higher drilling rate, voids
- loss of drilling water
- depth of detectable and substantial water ingress
- after drilling of a hole, drill string removed: Measure initial water ingress rate in l/min.

Apply maximum flushing with water and compressed air when pulling out the drill string at the end of drilling to facilitate efficient cleaning of the hole.

Grouting criteria, A: Injection can be carried out if any of the following criteria are met:

- Initial ingress from any single probe hole > 3 l/min.
- Total initial ingress from all holes > 6 l/min.
- Loss of more than 50 % of the flushing water (approximate) in any single hole.

Distance criteria, B: If all or a major part of the recorded ingress or loss of flushing water locations occur deeper than 15 m into the holes, then the face should be advanced to a minimum distance of 5 m from these features.

Step II: Grout filling of probe holes. Place a packer at a minimum of 2 meters into the probe holes and inject grout for to fill the hole. Stop if a pressure of 20 bar is reached, or if the pumped quantity reaches 300 kg. Holes can alternatively be filled by anchoring mortar through an open plastic hose from the bottom upwards.

Step III: Advance the face. Advance the face until a minimum of 5 meters of probing overlap is reached. Execute next stage of probe drilling.

Step IV: Add boreholes for grouting. Add boreholes to a total number of 8. Positions 6-9-12-3 and 7:30 and 4:30 should be covered first. The last two holes should be drilled in the area of most water ingress or flushing water loss.

Step V: Advance the face. Advance the face to a minimum distance of 5 meters from the features that initiated the grouting decision.

Step VI: Add boreholes for grouting. Add boreholes to a total number of 8. Positions 6-9-12-3 and 7:30 and 4:30 should be covered first. The last two holes should be drilled in the area of most water ingress or flushing water loss. The length of the added holes should be adjusted to end at the same chainage as the previous holes.

Step VII: Pressure grouting. After packer placement at a minimum of 1.5 m depth, start grouting in the lower part and work upwards. All holes should be grouted. Stop the grouting of a hole if the pressure reaches 50 bar, or if the pumped cement quantity reaches 1500 kg.

Step VIII: Control holes. Drill a minimum of 4 control holes (after careful evaluation of the required minimum time for cement hydration), and increase to 8 holes if high grout takes occurred in most of the previously grouted holes. Adjust the location of control holes based on the distribution of grout takes and the location of recorded features in the ground.

Apply grouting criteria A on the control holes to then decide on the next step.

Step IX: Grout filling of control holes. Place a packer at a minimum of 2 meters into the probe holes and inject grout to the hole. Stop if a pressure of 20 bar is reached, or if the pumped quantity reaches 300 kg. Holes can alternatively be filled by anchoring mortar through an open plastic hose from the bottom upwards.

11. EXAMPLES OF RESULTS ACHIEVED

11.1. General

The term waterproofing is used when considering sealing of rock by grouting. A more correct term would be ground water control or conductivity control. The reason for this is that a 100% drip free and watertight tunnel cannot be guaranteed by pre-injection and post-injection methods, even with the most elaborate procedures. The tunnel can be made almost waterproof, but not entirely.

However, for a number of purposes it is possible to reach requirements, and in many cases with less effort and cost than most engineers would assume in advance.

Generally speaking, it is possible to reduce the water ingress into a tunnel to a few percent of its original volume using pre-injection with reasonable means. One must be aware that the extra cost of additionally improving the result from e.g. 95% to 99% cut-off can be higher than sealing off the first 95%.

When looking into a number of projects where this technology has been used, the technology has developed considerably over the time span covered by these projects. What was state of the art 15 years ago is standard and easily achieved today, and may even be improved on if the situation calls for it.

11.2. What is achievable?

Almost anything is possible if resources are unlimited. It is more relevant to use project examples with a focus on the local situation, comparing the required and achieved results.

How much relative and absolute improvement of the ground water ingress situation that can be achieved will depend on the hydrogeological situation i.e. primarily the characteristics of the jointing and the number of joint sets.

Improvement in the ground water ingress may be limited to around two orders of magnitude if the rock mass surrounding the tunnel is highly fractured. This can be regarded as a worst case scenario. The sedimentary rocks in Oslo, Norway, are of this type, with extensive jointing along 3 joint sets with spacing on the 10 mm and 100 mm scale. Under such conditions, and based on a large volume of water pressure testing of boreholes (before grouting situation) and retrospective analysis of water ingress into pre-injected, executed tunnels (after grouting situation), the general rock permeability without grouting was determined as $k=1 \times 10^{-7}$ m/s and the improvement achievable by pre-injection by cement and chemicals was given as an end result $k= 2.5 \times 10^{-9}$ m/s [11.1]. It should be noted that this reference was published in 1987 so most of the project results are over 20 years old.

In hard rock conditions such as granites, granitic gneisses and similar stiff and brittle rocks, there is no real limit to the relative improvement obtainable by pre-injection. There are a large number of examples showing water features ahead of the tunnel face that would certainly drown the tunnel if left untreated. The same features are subsequently tunneled through without major problems after executed pre-injection. (There are also examples of decisions or «gambles» to go ahead with excavating without pre-injection in spite of serious indications that there was a lot of water ahead, causing flooding situations). The main consideration is to keep a tight bulkhead of sealed rock between the water feature and the tunnel face at all times until all the rock ahead of the ongoing excavation has been properly injected (see chapter 7).

To take one example, the Bjoroy sub-sea road tunnel found itself in extremely difficult ground conditions, with several hundred meters of tunnel producing full water flow from all probe holes at up to 7 bar pressure (70 m below sea level). The tunnel frequently crossed areas with water filled joints that were typically more than 100 mm wide with zero filling material. After excavating through one injected section, an originally 400 mm wide open joint was recorded as being completely filled by micro cement [11.2]. This statement can be found on page 252, Item 4.1.a:

> «Without pre-injection of a discontinuity of this size, the tunneling would simply have been impossible».

The paper furthermore states on page 250, Item 3:

> «When encountering the zone with exploratory drilling ahead of the tunnel face at a distance of 8–10 m, several cubic meters of sand and silt were flushed into the tunnel through the boreholes (51 mm diameter) together with water ingress of about 200 l/minute. Hence, the untreated condition of the soil is assumed to have had a behavior like running ground».

With pre-injection of cement, micro cement and acrylate grout, the required level of water ingress for this tunnel was actually reached. A satisfactory ground stability to allow very careful excavation through the zone was also provided. See more details later in this chapter.

11.3. Comparing shallow and deep tunnels

11.3.1. Some shallow hard rock tunnels in Sweden

Stille [11.3] discusses the development from unstable cement grouts using OPC to stable and low viscosity grouts with micro cement. To illustrate what can be achieved in terms of leakage reduction by cement injection, the paper presents a line nomograph as shown in Figure 11.1. Use of the nomograph by starting at an assumed ground water ingress of 1500 l/min per 1000 m and drawing a line through an assumed target of 200 l/min per 1000 m after injection, indicates that this is a medium level complexity. This is a reduction of water ingress by 87%.

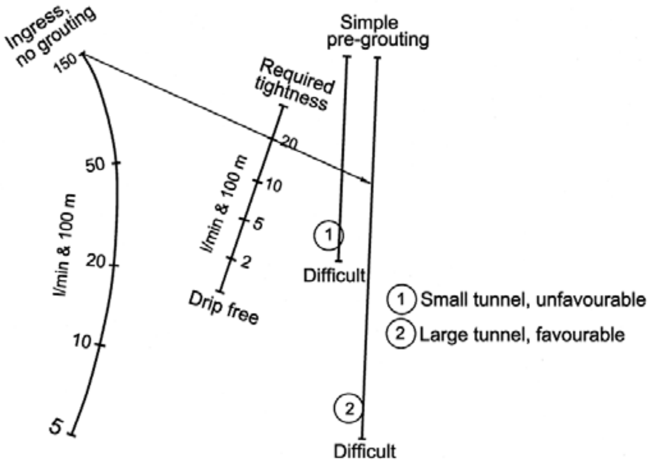


Figure 11.1 Nomograph indicating tightness target complexity

It is important to note that the difficulty of achieving a certain specified result in terms of final water ingress is far more dependent upon the required tightness level (50–100 or 200 l/min per 1000 m) than by the level of water ingress before any injection of the rock takes place. Even if the untreated ground would yield 15000 l/min per 1000 m this would not reduce the probability of reaching e.g. 200 l/min per 1000 m. Under hard rock conditions, reduction of water ingress by two orders of magnitude is frequently quite easy to reach.

Erikson and Palmqvist [11.4] report on specified water ingress limits between 0.5 and 2.5 l/min per 100 m, depending on local risk level in the project, as shown in figure 2 on page 161 of their paper. The measured water ingress after the end of the construction period showed results from 0.85 to 1.1 l/min per 100 m, as given in figure 7 on page 172. It is noteworthy that this result was reached with cementitious grouts only.

The English translation of the summary of the paper in reference [11.5] by Haessler and Forhaug reads as follows:

> «A good result can be achieved with close to no water drips from the roof in mica shist, even with relatively few curtains holes, fast grouting cycle and avoiding time consuming execution controls. Finely jointed

mica shist with clay-filled joints can be well grouted with stable grouts based on cement. An even curtain with small volumes in many holes can be better than an uneven curtain with large volumes in few holes. Extremely high pressure, especially in the first phase of the grouting, can improve the result. Development of grouting methods during the project is good for the result».

Sundin and Karlsson [11.6] report on a tunnel that is 3.7 km long and has a diameter of 3.5 m. The rock types are granites and granitic gneiss. Probe drilling ahead of the face was mainly carried out by drilling 8 holes, each with a length of 25 m, starting 8 m behind the face. The required maximum water ingress was 2 l/min per 100 m and pre-injection was successfully carried out where necessary.

Hahn [11.7] describes a tunnel which is 7.6 km long and has a diameter of 3.5 m. The rock types are granites and granitic gneiss with some amphibolite. The required maximum water ingress was 5 l/min per 100 m and pre-injection was carried out where necessary. A substantial part of the total water ingress originated from within 18% of the total tunnel length. About 15% of the total length of probe holes gave water loss measurements equal to or larger than 1.0 Lugeon (about 2×10^{-7} m/s). About 5% of the length showed more than 10 Lugeon (about 2×10^{-6} m/s).

On behalf of the Sodra Lenken highway tunnel project in Stockholm, the Royal Technical University of Stockholm sent out a material request to suppliers dated 12 May 1999 (Mr. T. Dalmalm). Some interesting information is given in the request in terms of the tightness requirements that were expected using pre-injection:

«Maximum allowed water ingress of 1 – 3 l/min per 100m.
Based on a number of tunneling projects in the Stockholm granitic rocks:
- 75 % of the rock mass has permeability < 1 Lugeon ($k = 10^{-7}$ m/s)
- 20 % is more jointed with $k > 10^{-6}$ m/s
- 5 % will cross shear zones

The cement injection material must satisfy the following:
- Shear strength > 3 kPa after 2 hours
- Bleeding maximum 2 % after 2 hours»

11.3.2. Some shallow tunnels in the Oslo area, Norway

Shallow tunneling in sedimentary, highly fractured rocks has been extensively carried out in Oslo. The tunneling in this area, all requiring ground water control by pre-injection, has exceed a total of 100 km in length. Some selected references from this area provide the following information:

Rock tunneling in the Oslo area requires pre-injection to avoid surface settlements in marine clay deposits. Some early experiences, such as the Holmenkollen subway commissioned in 1916, gave settlements up to 350 mm within 200–400 m from the tunnel alignment [11.8]. As stated on page 74, tunnels driven in Oslo's sedimentary rocks will generally yield in the range of 20 to 40 l/min per 100 m if not injected (this corresponds to an overall rock permeability of the order 10^{-7} m/s). To avoid surface damage in the most sensitive areas, pre-injection grouting must reduce the ingress to 1 to 2 l/min per 100 m. The authors also emphasize that post-grouting may be fairly successful in already pre-grouted areas, but post-grouting is stated to be no alternative to pre-grouting. The following statement can be found on page 75:

> «Experience shows that fairly good results from post-grouting can only be achieved in pre-grouted areas. Post-grouting is not an alternative to pre-grouting». Furthermore, «experience from recent road tunnels shows that water ingress may be reduced to 2 to 5 l/min per 100 m by the use of cement pre-injection in tunnels of 60 to 100 m^2 cross section».

11.3.3. Deep situated tunnels

Since 1979, many sub-sea road tunnels (some years ago it was already 20 of them) have been constructed in Norway. Most of them are located in hard rock, with a maximum depth of between 56 and 260 m below sea level, and all of them were systematically probe-drilled and pre-injected where necessary.

With cross sections in the range of 43 m^2 to 68 m^2, the water ingress after commissioning varies from 10 to 45 l/min per 100 m. These results have been achieved with cement grouting alone and with a targeted ingress limit of 30 l/min per 100 m. They are not illustrating what is ultimately achievable by injection techniques, but demonstrate well what can be achieved by using quite reasonable and limited measures.

11.4. Sedrun access tunnel, Alp Transit Project, Switzerland

The 1000 m long access tunnel to the vertical shaft (800 m down to the main tunnel level) hit a small sub-vertical shear zone that yielded about 200 l/min at 10 bar pressure. This concentrated ingress was not pre-injected and because of the nuisance of the flowing water, an attempt was made to reduce the ingress by post-injection.

As this was a concentrated ingress with good rock on both sides of the about one meter zone of disturbance, chances of succeeding with an acrylate grout were seen as acceptable. The contractor drilled injection holes that crossed the water channels, but at a depth of only one to two meters behind the tunnel contour. This was a communication mistake and was quite unfavorable for the execution of the work, due to the very short distance of grout backflow to the tunnel.

To counteract this situation the acrylate grout was prepared in batches, allowing a start of gel-formation before the start of the injection pumping (using an ordinary cement injection pump). In reality the pumping was done on gel lumps under formation (not liquid acrylate) that were still weak and soft. It turned out that these gel lumps started clogging up the backflow channels in the rock, and the ingress gradually decreased and finally almost completely blocked the flow channels.

The permanent residual ingress in this area has been measured at between 5 and 10 l/min, and this was satisfactory to the client, so no additional attempts were made at further reduction of the ingress (see Figure 11.2).

Figure 11.2 Acrylate grout appearing as limited backflow before sealing off

11.5. Bekkestua Road Tunnel, Oslo, Norway

The Bekkestua tunnel is 705 m long with a cross section of 68 m^2, and is located in a suburb of Oslo. The initiative to construct the tunnel was taken by the inhabitants of Bekkestua, who wanted to get rid of the heavy transit traffic through their town. The tunnel was excavated by the drill and blast method.

The rock cover consisted of between 2 and 50 m of highly jointed limestone with layers of shale. The rock support used consists of steel fiber reinforced sprayed concrete with sprayed-in steel arches in weak zones. As the tunnel is below ground water level with marine clay sediments resting on the bedrock, measures had to be taken to prevent drainage and lowering of the pore pressure in the soil. Surface settlement and damage would otherwise have been the result. The limit of water ingress into the tunnel was set at a maximum 2 l/min per 100 m tunnel length.

11.5.1. Practical execution in the Bekkestua Tunnel

A round of 25 holes were drilled per pre-injection station with a length of 21 m. Recorded water ingress measured at more than 5 l/min per hole was treated with normal Portland cement and 2% RHEOBUILD® 1000 admixture for water reduction. The maximum cement quantity per hole was set at 4000 kg and the maximum injection pressure at 30 bar. One must note the relatively high pressure used despite the rock cover being quite limited (no damages occurred).

For water ingress measured at less than 5 l/min per hole, injection was carried out with RHEOCEM micro cement with 3% RHEOBUILD 1000, a water reducing and dispersing admixture. The maximum micro cement quantity per hole was 2000 kg and maximum pressure 30 bar.

The resulting total water ingress to the tunnel at the end of the excavation period was measured at 0.7 l/min per 100 m tunnel. The largest leakage of 1.7 l/min per 100 m was recorded in a section where only OPC had been used (no micro cement).

The project consumed a total of 583 tons of RHEOCEM 650, 40 tons of RHEOCEM 900 and 556 tons of OPC. This was injected through 1440 packer placements and distributed over 26000 m of boreholes. The quantities are comparatively high, but this is linked to the very strict ingress limit and the highly jointed sedimentary rocks. Execution took place from August 1993 to March 1994. See also chapter 11, Figure 11.2, which illustrates the efficiency of using RHEOCEM microfine cement in regard to time spent, quantities injected and the final result.

11.6. The Bjoroy sub-sea road tunnel

11.6.1. The project

The 1965 m long Bjoroy road tunnel passes under the strait of Vatlestraumen near the city of Bergen in SW Norway. The tunnel reaches a maximum depth of 80 m below sea level. Excavation began in November 1993 on the island side. Breakthrough was reached in August 1995, when 840 m had been excavated from the island side and 1125 m from the mainland. The tunnel was opened to the public in 1996.

11.6.2. The challenge

Extreme conditions were encountered after about 700 m of excavation from the Bjoroy side. During routine probe drilling ahead of the face, flowing sand and silt under 7 bar water head was hit at 8 to 10 m in front of the face. Within a few minutes, several cubic meters of water and sand had blown into the tunnel through one single 51 mm diameter borehole.

The hole yielded water at about 200 l/min.

The main part of the fault system encountered turned out to be a Jurassic formation with competent sandstone, sedimentary breccia and unconsolidated sand and silt. The thickness of loose sand material varied from a few cm to 2.5 m, while the complete zone had a maximum thickness of about 4 m. The tunnel crossed the zone at 72 m below sea level with rock cover of about 30 m (see Figure 11.3).

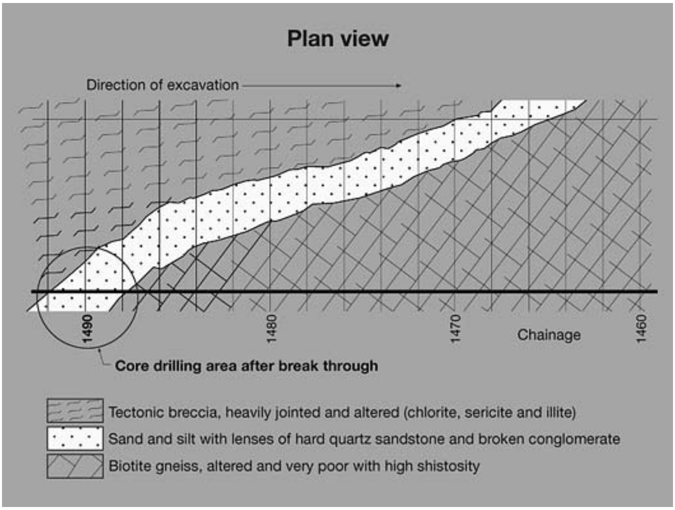


Figure 11.3 The zone with running ground

It was quickly agreed that to enter into this type of flowing ground with a tunnel face about 60 m^2 in cross section without taking special precautions would be impossible. A number of different technical solutions were considered, including ground freezing, horizontal jet grouting and different types of spiling and micro pillar installation. To be able to use ground consolidation by pressure pre-injection, it was necessary to ensure sufficient permeation into the silty soil to create the necessary water cut-off and sufficient ground stability improvement.

11.6.3. The solution

Extensive ground consolidation activities were undertaken in order to improve ground stability, allowing an open face excavation and support. Ground consolidation techniques included cement based compaction and hydrofracturing grouting, chemical acrylic hydrofracturing and permeation grouting, as well as gravity drainage from the zone.

Support ahead of the face by spiling and immediate sprayed concrete support after short excavation steps was used. Stability was monitored systematically by the use of convergence measurements.

Key elements of the chosen solution were the quick setting, high strength ultra fine micro cement RHEOCEM 900 and the acrylate resin MEYCO MP 301. The resin provided a permeation capability in the fine sand and silt material and created a simultaneous sealing and strengthening effect in the injected ground (see Figure 11.4). The cement was always used first in several stages, until the necessary homogeneity was achieved to allow pressure build up and permeation into the sand lenses by the acrylate resin.

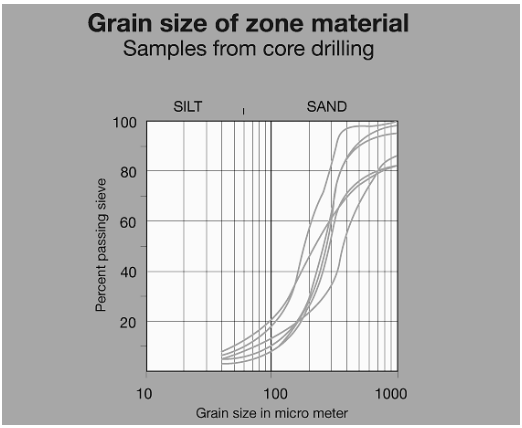


Figure 11.4 Zone material sieve analysis

Injection was done through steel standpipes placed around the tunnel contour. These pipes also had the function of micro piles for the subsequent excavation. Excavation by backhoe in very short steps and on a partial face area only was followed immediately by steel fiber reinforced sprayed concrete.

11.6.4. Results

The progress through the circa 30 m of tunnel length, directly influenced by the zone, proceeded slowly and successfully.

Only minimal water seepage was observed and there were no blowouts or uncontrolled collapse areas. A research program carried out by the inspection of cores drilled through the final concrete lining after the tunnel break through can be summed up as follows:

- Of the ground sampled and inspected, about 50% consisted of compacted silty sand. The compaction effect was sufficient to produce core recovery.
- About 25% of the ground did not allow core recovery.
- Of the silt and sand material, 10 to 15% had been permeated by the acrylate resin grout MEYCO MP 301.
- Cement lenses had replaced 10 to 15% of the sand/silt ground by splitting, causing compaction of the adjacent silt material.
- Silt permeated by MEYCO MP 301 showed compressive strength of 0.36 and 0.39 MPa.

For a more complete presentation of the project see [11.6].

11.7. The Ormen Project, Stockholm, Sweden

11.7.1. The project

With a frequency of circa once every 5 years, heavy rainfall hits the city of Stockholm. This used to cause problems as the capacity of the network of pipelines for rain and waste water drainage was insufficient. To reduce overflows into the surrounding rivers and lakes, a tunnel was excavated to serve as a temporary storage of surplus water until the demand on the pipelines and the waste water treatment plants was reduced.

Eight raise-bored shafts lead the rain water from the streets down into the tunnel. The tunnel got its name (the Snake) due to its winding form. The Snake was constructed at a depth of 40 to 60 m between the central parts of Stockholm city in an extremely sensitive area of the old town where many of the houses are supported on wooden piles. Any lowering of the ground water level in the vicinity of these buildings would have resulted in serious settlements, rotting of the piles and damage to the buildings.

For this reason the level of maximum permitted ingress of water into the tunnel was set at 2 l/min per 100 m tunnel. This is a very strict requirement, which means an almost dry tunnel.

11.7.2. Tunnel data

The tunnel mainly passes through crystalline gneisses (50%) and granites (40%), interspersed with zones of fractured and weathered rock (10 %).

The tunnel diameter is 3.5 m, the total length is 3700 m and it was excavated by a TBM. Average tunnel production including pre-injection works was 15 m per day.

In order to meet the project requirements it was decided to use a TBM to eliminate the risk of vibration damages to overlying structures, as well as to reduce the risk of extra water ingress caused by blasting cracks in the surrounding rock.

Continuous pre-injection along the whole tunnel alignment was necessary in order to seal cracks and joints to keep water ingress below the specified limit. RHEOCEM micro cement was selected as the grout material for this work.

RHEOCEM requires a modern colloidal mixer, and high pressure should be used during injection (from 30 to 60 bar). The selected pump from Montanbuero therefore had a working pressure of 100 bar. The equipment worked reliably throughout the project.

In this case, the MEYCO team worked as consultants to the contractor (Siab), and produced working guidelines and procedures. MEYCO was also involved in tendering, and further assisted the contractor in discussions with the owner. Both the theoretical and the practical training was conducted by MEYCO.

11.7.3. Some general information

Siab AB injected 160 tons of RHEOCEM 650 and 40 tons of RHEOCEM 900. The construction period was from February 1991 until June 1992.

11.8. Limerick main drainage water tunnel, Ireland

11.8.1. The project

The tunnel provides a new drainage system for the city of Limerick, linked to a state of the art sewage treatment plant at the downstream end. This eliminates all untreated discharge to the river and is an important environmental improvement.

Murphy Tunneling was the contractor for the 2550 m of 2.82 m inner diameter EPBM drive. The tunnel is lined with concrete segments and runs at about a 15 m depth. Access for excavation and for sewage connection points are through 13 vertical shafts.

11.8.2. The challenge

One of the access shafts down to the main tunnel was located in water bearing fine sand and this soil needed stabilization to allow safe break-in and break-out of the TBM at the shaft. The soft alluvial deposits, the high water head and the proximity to the river added to the construction problems. Figure 11.5 illustrates the general layout of the shaft and tunnel.

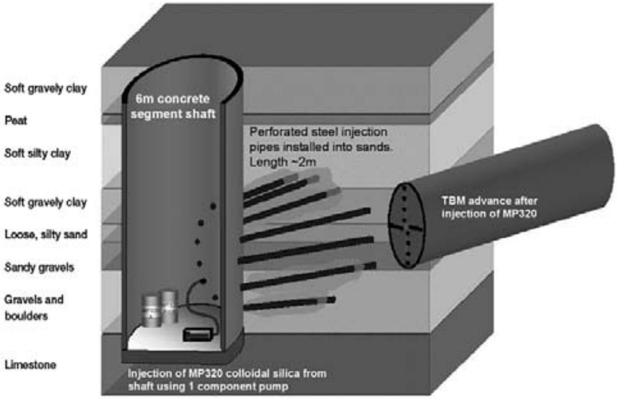


Figure 11.5 Break-in area soil stabilization

11.8.3. The solution

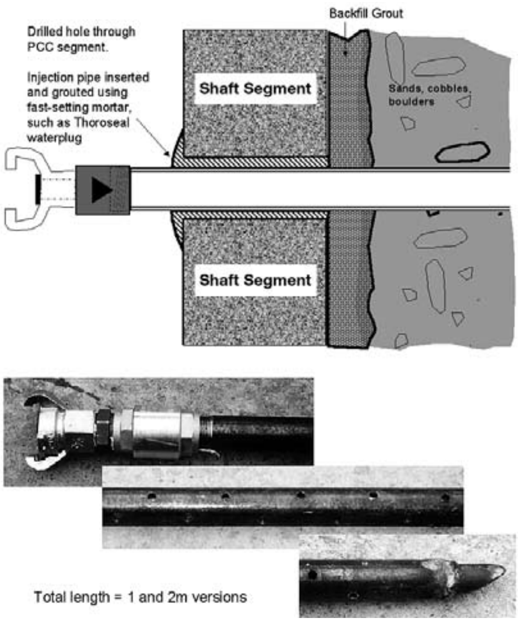


Figure 11.6 Injection point through shaft segment lining

Fine sands and silt will typically cause injection problems using cement or even ultra fine cement. The penetration may stop prematurely, and it is very difficult to achieve a uniform product distribution (and effect). Many chemical resin products can cause environmental hazards in certain locations, and it was therefore suggested to grout with MEYCO MP 320 colloidal silica gel.

The colloidal silica is a nanometric sol, with a primary particle size of 0.015 micron, coupled with a viscosity of 5 mPas (cP) (similar to skimmed milk). This product will penetrate fine sand and coarse silt.

To facilitate injection, a series of one and two meter long perforated steel pipes were rammed into the sand through pre-drilled holes in the shaft segment lining. Positions were marked around the circumference of the approaching TBM break through. The pipes were sealed in place by quick setting mortar (see Figure 11.6).

The MEYCO MP 320 was pre-mixed with 20% of component B (10% solution of table salt in water), giving an open time of 30 minutes. This open time allowed sufficient permeation distance for consolidation and the use of standard one-component cement injection equipment.

11.8.4. Results

The mixing of MEYCO MP 320 proved very simple to undertake, and the use of standard cement equipment offered a clear advantage to the tunnel crew carrying out the grouting, as they were already familiar with this equipment.

After removal of the shaft segments to allow the safe break through and break out, the sand was seen to be effectively treated and stable. The MEYCO MP 320 had solved the problem of ground water control and stability of the soil without causing any problems for the TBM.

11.9. The Kilkenny main drainage tunnel, Ireland

11.9.1. The project

The tunnel is circa one meter in diameter and 200 m long, driven by pipe jacking. It passes beneath the town center with between 5 and 10 m cover to the surface. The tunnel was driven through fine to silty saturated sand, causing considerable construction problems.

11.9.2. The challenge

The sand was saturated with water, causing it to flow readily once exposed during excavation. Consequently, the original traditional pipe jacking method was abandoned for an Iseki micro-tunneling machine with jacked steel pipes. There were still considerable problems, like one occasion where the head was almost lost to an oversized wash out cavity in the ground. Also, settlement problems occurred due to the close proximity to the foundations of old town buildings along the route.

Various ground treatment systems had been used to improve the stability of the sand, including PFA, Bentonite and cement injection, water-glass injection as well as jet grouting. None of the systems accomplished any improvement in the tunneling conditions.

11.9.3. The solution

When about 10 m remained to complete the tunnel drive, the sand demonstrated worsening instability. The Iseki machine could therefore not achieve the steering accuracy needed to reach the target in the reception chamber, constructed with pre-cast concrete rings.

Samples of the sand gave a particle distribution between 0.063 mm and about 2.0 mm, with roughly 95% smaller than 1.0 mm. This indicated soil conditions well within the range of the ground treatment envelope offered by MEYCO MP 301 acrylic grout, and on the lower border of what is possible with RHEOCEM 900 ultra fine cement. For cost reasons, the contractor wished to try RHEOCEM 900, but finally used the acrylic grout as this clearly offered the best solution.

Injection pipes were inserted from the concrete segment reception chamber in a horizontal umbrella fan arrangement. Pipe spacing was approximately 300 mm. Low pressure injection was carried out using a hand pump system, and the MEYCO MP 301 acrylic resin was mixed 1:1 with the B component.

11.9.4. Results

During excavation of the final 10 m of the tunnel, the Iseki machine was able to continue with improved steering control. The continuous sand washout experienced prior to injection of the acrylic grout was now stopped. Some clear water was running on the invert of the tunnel, whereas before the invert was filled with silt and fine sand. Surface settlement was also well controlled as a result of the grouting.

11.10. West Process propane cavern project (WPPC), Norway

11.10.1. The project

As an addition to the existing oil and gas facilities at Mongstad, north of Bergen, a rock cavern was constructed for the storage of liquefied propane gas. The actual rock cavern is 33 m high, 21 m wide, and its length is 134 m. The floor of the cavern is located 83 m below sea level to allow for ground water head larger than the gas pressure above the liquid propane.

To allow the storage of propane in liquid form, the gas has to be stored at -42°C. The freezing down of the rock surrounding the cavern was started by air circulation, and at the end by filling it with liquid propane.

Figure 11.7 The propane storage cavern at Mongstad, Norway

11.10.2. The challenge

For such unlined gas storage to function properly and avoid gas leakage to the surroundings, it is crucial to maintain the ground water level during all stages of construction and operation. This was achieved by systematic pre-excavation grouting and by the installed water infiltration system. About 4000 m of guided boreholes were drilled above the cavern for this purpose.

To be able to carry out the freezing down of the surrounding rock it was also necessary to limit the water ingress. Flowing water would otherwise transport heat into the cavern and at concentrated water ingress spots it could become impossible to stop the water from ice building. It was estimated that the ground water ingress would have to be less than 15 l/min measured over the whole cavern.

11.10.3. The solution

A program was developed for systematic pre-grouting of all excavation stages (top heading, benches and invert). All the grouting was done by RHEOCEM 900 ultra fine cement with RHEOBUILD 2000 PF at 1.5 % by weight. The w/c-ratio of the grout varied from 0.8 to 1.0 by weight.

The pre-grouting work required about 30000 m of boreholes and consumed 410 tons of cement.

11.10.4. Results

The total ground water ingress after the end of excavation amounted to less than 2.0 l/min with the ground water level being virtually undisturbed by the project.

Some grouting had to be done in the 450 m of vertical shafts (diameter of 2.1 m). The shafts, which have steel lining with concrete backfill, were being affected by water trickling through the rock/concrete contact. To stop the water at the bottom part, 400 kg of MEYCO® MP 355 1K polyurethane foam was used. After this blockage was in place, a total of 500 l of MEYCO MP 320 colloidal silica was injected. The injection hoses in the steel/concrete contact were also injected by MEYCO MP 320.

11.11. Recent project result

To put the modern high-pressure approach with micro cement and its potential achievements into perspective (which would be impossible with traditional low-pressure grouting and bleeding OPC grouts), one can look at the following project comparison:

Danilo Abdanur and Carlos Alexandre de Almeida presented a paper at the International Symposium on Waterproofing for Underground Structures, Sao Paulo, Nov. 2005, with the title:

IMPERMEABILIZAÇÃO DOS SISTEMAS DE ANÉIS SEGMENTADOS ESTUDO DE CASO – ANEL DE CONCRETO LINHA 4 DO METRÔ DE CARACAS

The TBM-tunnel concrete segments were gasketed, backfill grouting of the annular space was carried out, and PU-post-injection was used where visible water ingress occurred. The end result was:

1–2 liters/minute per 100 m tunnel, which is an almost dry tunnel.

We move to the Asker–Jong tunnel outside Oslo, Norway for comparison:

- A twin track D&B railway tunnel (104 m^2 cross section)

- Highly broken sedimentary rocks interlayered limestone and claystone with igneous dykes
- Systematic pre-excavation grouting was executed. Sprayed concrete was used for permanent lining.
- Hydrostatic GW head up to 2X Caracas
- (if no grouting: Typically 20–50 l/min per 100 m tunnel would result, with even more at local igneous dykes (experience data))
- Quantity injected: 2 500 tons RHEOCEM 800, micro cement

The result achieved:
Less than 2 l/min and 100 m at double the hydrostatic head of the Caracas tunnel, and with a larger cross section. Anything like this would have been considered impossible only a decade earlier.

11.12. Oset drinking water treatment plant, Oslo, Norway

11.12.1. The project

The Oset Drinking Water cleaning plant is situated in Maridalen, Oslo. Client: Oslo Kommune, Contractor: AF Spesialprosjekt A/S and Krüger A/S JV.

The plant is built in hard rock with 2 caverns (100000 m^3) and a 500 m long tunnel. Total excavation amounts to 140000 m^3 rock. The cleaning plant is designed to treat 390000 m^3 water per day, and delivers drinking water to about 500000 people. (see Figure 11.8).

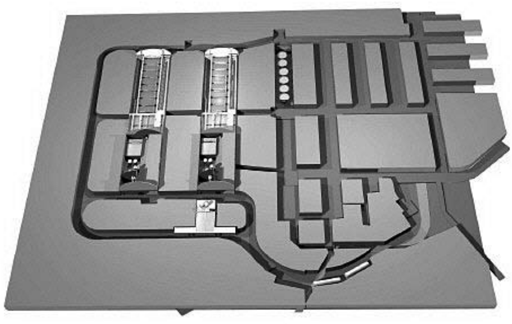


Figure 11.8 Layout of Oset water treatment plant

11.12.2. The challenge

The plant is located in syenitic rock of good quality. The average Q-value was 40, but there were also weak zones with Q-value <1. The allowed water ingress was set at 100 l/min for the whole plant. During construction, water ingress was measured at up to 200 l/min in some of the investigation probe holes.

11.12.3. The solution

The whole project, tunnel and caverns were systematically pre-injected with RHEOCEM 650 (micro cement) and supplemented with rapid hardening OPC. For some zones, accelerated grout (RHEOCEM 650 + MEYCO SA 162 alkali free accelerator) was used. The length of the injection holes was 21 meters, with a hole spacing between 1.5 –2 meters. After the injection, 3 rounds of advance were made, making the injection overlap about 6 meters.

Injection was carried out with a modern computerized injection rig. The rig had an integrated accelerator dosage pump for the injection of accelerated grout.

Figure 11.9 Cavern excavation

11.12.4. Results

The material consumption amounted to 1510 tons of RHEOCEM 650, 820 tons OPC and 38 tons of accelerated RHEOCEM grout used for blocking backflow or limit material spread.

The final result was unexpectedly good. The total ingress on the whole plant (tunnel and caverns), is only 20 l/min (the requirement being 100 l/ minute).

11.13. Arrowhead tunnels in Ontario, California, USA

11.13.1. The project

The 13 km tunneling project was built by the JV of J.F. Shea and Kenny Construction and completed in 2010. The tunnels are located near the base of the San Bernardino Mountains (East of Los Angeles), which are crossed by several significant faults. The distance to the well known San Andreas Fault is just 1 km, and the East Tunnel from the Strawberry Creek portal in particular crosses some extremely poor ground. Static groundwater head of up to 20 bar (300 psi) was experienced, and when combined with soil-like weathered rock, the overall mining conditions became adverse. An essential key construction requirement was to protect the groundwater resources during tunnel construction. The U.S. Forest Service Special Use Permit limited the groundwater inflow into the tunnel heading of the East Tunnel to a maximum of 2000 l/min, and this limit did not represent a problem under "normal" ground conditions. However, to keep the groundwater ingress below the allowed limit was only part of the problem when the tunneling hit adverse conditions with flowing ground at high pressure.

11.13.2. The challenge

Under most hard rock conditions, it would not be difficult to satisfy the stated water ingress limit by simply using Portland cement and perhaps microfine cement. This would apply even if sections of the tunnel showed heavily broken ground. However, in the Arrowhead East tunnel, the extensive use of ultrafine cement faced serious problems in achieving the necessary penetration and distribution to allow proper blockage of groundwater flow. The ground response was quite unpredictable as some bore holes took cement grout whilst others took close to nothing. This happened regardless of initial water yield, from just a few liters per minute to more than 700 liters per minute. Even if a bore hole took several thousand kilograms of cement, drilling new holes close by existing ones could still hit water as if no grouting had been carried out at all. The primary reason for the described ground response to grouting was a variable degree of in-situ weathering and decomposition of the granitic bedrock. When injecting cement grout into highly weathered ground, the front of grout material would pick up the fines (silt/sand) while spreading into cracks and joints, and a filter would form creating a blockage. The problem of groundwater ingress control turned into a problem of stabilizing and controlling flowing ground. In some locations the granitic rock matrix was so weakened and porous that the high hydraulic gradient caused by drilling a borehole resulted in local hydraulic collapse. The rock material turned into sand and blew out of the hole as if from a fire hose. If insufficiently grouted, subsequent mining into the area could lead to face or periphery collapse and bring the excavation to a halt, sometimes for weeks.

11.13.3. The solution

The solution was the use of colloidal silica MEYCO MP 320 T where the ultrafine cement did not sufficiently penetrate and seal off the ground. Colloidal silica is a water suspension of nanometric sized silica particles. The particle size is about 1/10th of the particle size of cigarette smoke. Gel time can easily be selected from a few seconds to more than 2 hours by varying the accelerator dosage. The product has a very low viscosity (5 cP) and it is well suited for all situations where penetration into fine cracks, joints and pores is necessary and difficult to achieve with particulate grouts. The gelling behavior of colloidal silica is very favorable as low viscosity is maintained until the preset gel time, when the viscosity then increases rapidly.

The strength of the gel is good and there is no syneresis or shrinkage when used in moist surroundings underground. This creates very good water tightness and the ground streng-thening effect is also noticeable in loose soils, running sand and in very broken rock.

11.13.4. Results

The Arrowhead East Tunnel has passed some extremely variable ground, and the worst section is probably the most difficult any machine of its kind has mined through.
The “cuttings” emerging on the conveyor belt during mining looked like silty sand. The successful tunneling through such adverse conditions must be attributed to a combination of several important factors:

- A highly competent contractor and dedicated tunneling staff working in close cooperation with involved consultants to develop the technical solution
- A custom designed TBM that firstly allowed the drilling of a sufficient number of bore holes to adequately cover the tunnel periphery with positions for probing and pre-excavation grouting
- The colloidal silica that allowed permeation into the part of unstable flowing ground where ultrafine cement could not penetrate adequately, thus completing the grouting coverage and preventing local blow-outs

We cannot know for sure how the “extreme” zones would have been crossed without the supplement of colloidal silica. It remains that colloidal silica turned out to be an important element of the solution, and that it helped speed up the tunnel advance and produce a good result.

11.14. Deep Tunnel Sewerage System T-06, Singapore

11.14.1. The project

The Queensway Tunnel (T-06) is part of the Deep Tunnel Sewerage System (DTSS) which the Singapore government commissioned to serve the wastewater transport, treatment and disposal needs for the 21st century. The T-06 main contractor was Züblin. The tunnel drive is 9.6 km with a finished diameter of 3.3 m.

11.14.2. The challenge

High water seepage through the sprayed concrete lining of the NATM tunnel (40 m long, 6 m diameter) from the launch shaft before the bored tunnel section occurred. The water seepage had to be addressed before the cast in-situ concrete lining could be placed in the NATM section. The tunnel is approximately 40 m below ground surface.

Polyurethane grout had been tried but was not effective to seal off the water as it was unable to penetrate the soil. It merely moved the water from one place to another. Cement injection was also tried but it could not penetrate the soil either and was therefore unsuccessful. The soil was mainly composed of silty sand.

11.14.3. The solution

MEYCO MP 320 is a mineral-based nanometric grout with a penetration capability similar to water, allowing permeation into fine sand and coarse silt. As the chemical base is SiO_2 and water, this offered the contractor a safe product for the tunnel operatives and the environment.

To facilitate injection, short packers were rammed into the soil through pre-drilled holes in the sprayed concrete lining where there was water leakage. The packers were sealed with the quick setting mortar waterplug. The MEYCO MP 320 was pre-mixed with 20% of component B, giving an open time of the gel of approximately 30 minutes. This open time allowed for the gel to be pumped through standard cement grouting equipment and enabled it to effectively permeate the silty sand. A maximum pumping pressure of 5.5 bars was observed throughout the whole injection process. The setup and equipment that was used is shown in Figure 11.10, and consisted of product barrels and a double diaphragm pump.

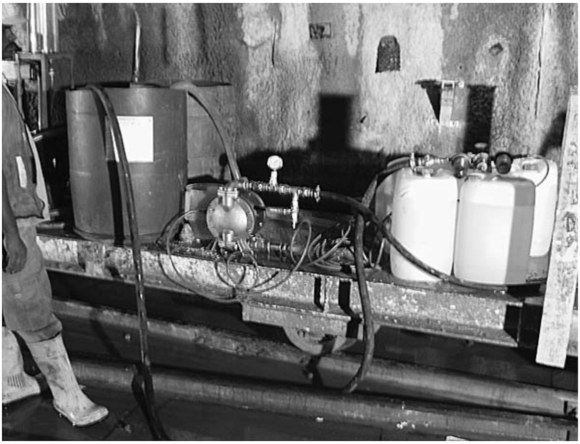


Figure 11.10 Equipment setup

11.14.4. Results

The MEYCO MP 320 colloidal silica gel penetrated the silty sand very effectively and allowed rapid sealing of the sprayed concrete NATM lining, thus allowing the construction of the cast in-situ concrete lining under dry conditions.

The simplicity of the product handling allowed the work to be initiated and completed with minimal disruption to other activities, and the problems of managing chemical grouts was avoided.

11.15. High speed railway Naples-Milan: Bologna City underpass

11.15.1. The project

The new high speed rail line from Naples to Milan crosses under Bologna through a 10 km long tunnel. In this area there are sensitive urban developments on the surface which require the tunneling works to minimally impact the environment. The contractor, S. Ruffillio Scarl, used a Lovat EPBM for this purpose. Two parallel tunnels were constructed, and every 250 m there are cross-passages with a cross section of about 12 m^2 and variable lengths between the two main tunnels.

11.15.2. The challenge

The hydrogeological situation in the ground was heterogeneous. After approximately 1000 m of clay from the portal of the tunnels, gravel sand and silt were found. These soils had different permeability properties. The cross passages were constructed by traditional excavation methods, with a pre-consolidation with normal cement based grouts, followed by excavation of the sand. The coarse sand and gravel were easily injected with cement based grouts. However, in the fine sand containing silt, with much lower permeability, the cement based grouts did not permeate as required.

11.15.3. The solution

With very low permeability in the silty sand, the only feasible grout alternative was colloidal silica, MEYCO MP 320. The grout suspension holds particles with a size of 16 nanometers, and the viscosity of the grout is just 5 mPas. It was found that the grout could easily permeate the silty sand.

By using a simple one-component pump, a total quantity of about 45000 kg of MEYCO MP 320, with 10% accelerator dosage was injected into two cross passages. This accelerator dosage gave a gelling time of about 150 minutes. Pumping pressure was in the range of 2–5 bar (see Figure 11.11).

Figure 11.11 Cross passage excavation after injection

11.15.4. Results

The resulting improvement of stability in the silty sand made traditional excavation of the cross passages quick and easy, without any problems of collapsing ground (see Figure 11.12). Based on this experience, the contractor decided to also apply this technology in the remaining cross passages where silty sand is prevalent.

Figure 11.12 Stabilized soil

11.16. The Ghomrud water tunnel project, Iran

11.16.1. The project

The 36 km long Dez-Ghomrud water diversion tunnel in western central Iran is divided into four main contracts. The last 8 km of the tunnel heading downstream is located in a mica-schist with generally good stability. This part was excavated with a double shielded, 4.6 m diameter hard rock TBM. The trapezoidal segment lining with pea gravel annulus fill was installed concurrently with the excavation. After approximately 3.2 km of excavation, the TBM encountered a weak rock graphite schist with poor stability. When attempting to excavate through this ground, several collapses occurred over the TBM cutter head and shield. The collapses propagated up to 8–10 m height over the TBM.

11.16.2. The challenge

The cavities that formed above the cutter head represented an imminent risk of continued deterioration of the stability of the rock mass above the TBM. A temporary measure to fill and stabilize the cavities was therefore needed. This task could be broken down into the following elements:

- Filling of the voids over the TBM by means of injection
- Possibility for stepwise application enabling complete filling of the voids from different working positions on the TBM
- Immediate application to resolve this urgent and difficult situation
- Controlled setting and placement of the grout to avoid large volumes of unreacted or unhardened grout flowing back into the TBM space

11.16.3. The solution

Injection of rapid foaming polyurethane was suggested. The expanding polyurethane would create a complete filling of the voids, as well as consolidating the rock debris which was resting on top of the shield and the cutter head. The rapid reaction of the foam would cause it to remain in place where intended, and not flow into the TBM space. For the sake of availability, both the one-component polyurethane MEYCO

MP 355 1K and the two-component polyurethane MEYCO MP 355 / A3 were used. A two-component high pressure pump was used for both products. In order to obtain the highest possible expansion factor of the polyurethane, water had to be added as the ground was completely dry. For the two-component polyurethane, water was added to component A (1% by volume), hence creating an instant foam when the two components were mixed. For the one-component polyurethane, water was sprayed into the void through perforated plastic pipes. This created the required wet conditions, and the polyurethane foamed instantly (see Figure 11.13).

Figure 11.13 Injection of PU

Time is obviously important in such cases. Equipment and products were both on stock in Tehran and were delivered on site 7 hours after the site management made their decision to inject. Application was performed with the specialist contractor Paya Beton with support from MEYCO.

11.16.4. Results

Approximately 2 tons of polyurethane was injected over the TBM. This gave a total foam volume of approximately 35 m^3 so a significant void was filled. The TBM was moved forwards and the segments could be placed without debris caving into the tunnel.

11.17. River Aare underpass, Bern, Switzerland

11.17.1. The project

A gas duct for the municipality of Bern was to be placed in a tunnel passing under the river Aare. The river underpass is located in the middle of the city. The ground conditions consist of marl limestone covered by a 6 – 8 m thick soil layer of gravel and sand. A shaft on each side of the river was constructed, employing piles which were rammed to the rock surface, allowing for excavation down to the level of the rock surface. The excavation of the lower part of the shaft (located in the marl) was done by a mechanical excavator (roadheader). Seepage into the shaft should be as little as possible for obvious reasons. The situation can be seen in Figure 11.14.

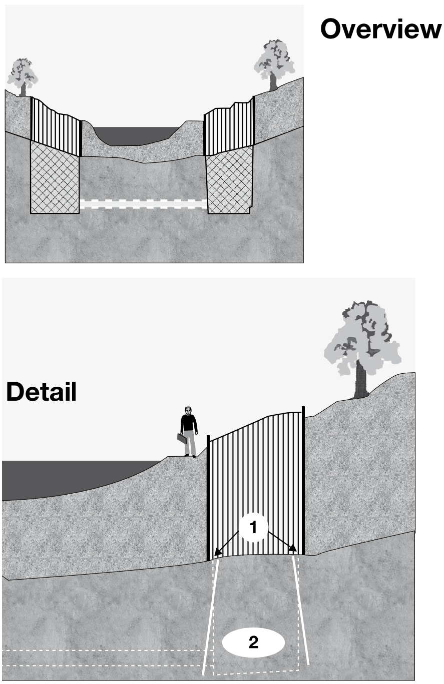


Figure 11.14 Longitudinal section.
1) PU injection
2) Colloidal silica injection

11.17.2. The challenge

The contractor's goal was to seal off the water seeping in along the lower part of the pile wall. Subsequently, he wished to prevent water ingress through the marl rock mass when excavating the lower part of the shaft into the rock. Hence, a pre-injection procedure with vertical drillings from the bottom of the soil excavation was laid out. The following criteria were essential to the contractor:

- Cost-effective and rapid reduction of water ingress
- Availability: Immediate and rapid execution of the works
- Environmentally safe: No use of toxic components or negative visible effects in the vicinity of the injection works

11.17.3. The solution

The use of the one-component polyurethane, MEYCO MP 355 1K was suggested for the cut-off of water along the lower part of the pile wall. The product was injected through 25 mm steel pipes which were rammed into the ground through small gaps under the pile wall. The following features of this method and product properties were favorable:

- High foam factor in contact with water, even at the relatively low temperatures encountered at this site (-1 to +2°C)
- The simplicity of the overall application
- The result was immediately visible, and additional injection could be decided on and executed immediately

For the pre-injection of the rock part of the shaft, colloidal silica, MEYCO MP 320 was used. This offered the following advantages:

- Excellent penetrability (far better than micro cement) in the water bearing fine joints of the marl
- Very accurately adjustable gelling time
- Possibility to speed up the gelling and inject the gel in a stiff state in the event of backflow
- Absolute environmental friendliness

11.17.4. Results

When excavating the lower part of the shaft in the marl limestone, one could observe fine veins and lenses of colloidal silica in the rock mass. No significant water ingress occurred.

11.18. Maneri Bhali Phase II hydropower project, Himalaya

11.18.1. The project

In the 90's, the excavation of the original headrace tunnel encountered a heavily faulted zone with an overlying river valley where rock cover decreased to only a few meters, with overlying saturated river borne deposits. At the time of the original excavation, cave-ins and severe water ingress were encountered in this zone that led the tunnel to collapse, resulting in heavy delays to the project [11.9].

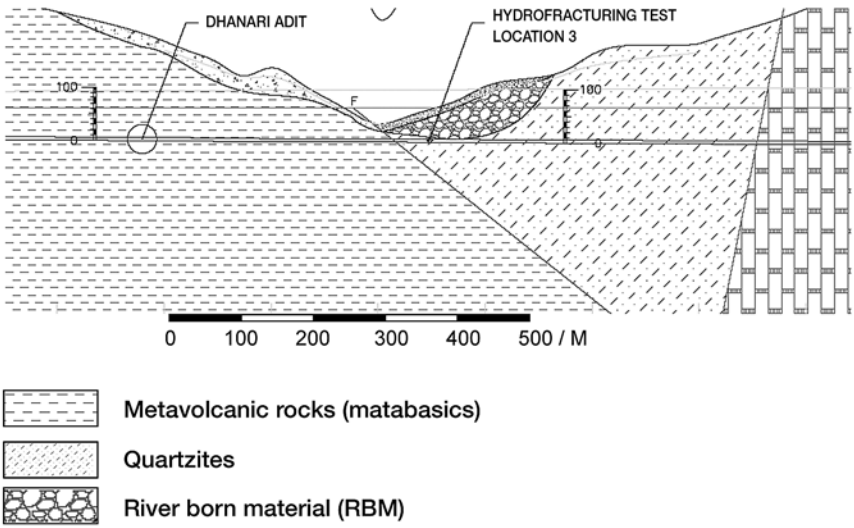


Figure 11.15 Longitudinal section through the tunnel and shear zone

On recommencement of the project in 2003, a bypass tunnel was proposed to reconstruct the tunnel in this section. This would allow proper construction and waterproofing of the new section of tunnel across the fault zone, whilst allowing excavation and lining work to continue unobstructed in the main tunnel.

11.18.2. The challenge

The challenge was again to cross this fault zone in the bypass tunnel, without ground collapse and in dry conditions. The rock mass consisted of heavily jointed quartzite and metabasic rocks on each side of a regional fault zone. The river valley on the surface followed the fault zone alignment with thick deposits of river borne materials overlying the quartzite and metabasics.

The fault zone exhibited highly crushed material with associated high water seepage that resulted in significantly reduced stability of the excavated rock mass. Joint fillings consisted of fine grained quartzite material of clay and silt fraction.

Initially, the tunnel was excavated through this zone without any pretreatment of the rock mass. Large water ingress and cave-ins occurred. A very irregular tunnel contour was the result. The tunnel was supported with steel sets and concrete lagging. Backfill of concrete was attempted but only with limited success. The major part of the water ingress remained.

Therefore it was decided to excavate a bypass tunnel around the problem area. Pre-treatment ahead of the excavation face was decided on in order to improve the geomechanical properties of the ground, as well as to reduce the water ingress to a minimum.

11.18.3. The solution

The first attempts at pre-injection with ordinary Portland cement (OPC) were not successful. Penetration into the ground was not achieved and in most cases, only the filling of the drill holes with grout was achieved.

After the initial attempts also using micro cements, it was obvious that a two-stage injection scheme would be required. This consisted of a first stage injection with micro cement.

Following the first stage micro cement injection, a few holes were drilled ahead of the tunnel face to verify the achieved water seepage reduction and improved properties of the rock mass (by drillability). If the result did not prove satisfactory, second stage injection would be necessary.

The second stage of injections was designed as an inner injection fan with colloidal silica (MEYCO MP 320 T), which was entirely covered by the rock mass volume treated under the first stage of injections. Figure 11.16 illustrates the layout of the two stages of injections.

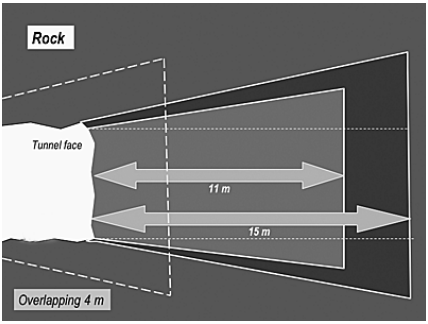


Figure 11.16 Two stage injection (colloidal silica in the light-colored areas)

With this method it was possible to advance the tunnel face approximately 8 m before a new injection cycle was necessary.

The jointing of the rock mass caused unstable boreholes, and the feasible drilling length was therefore limited. It was also difficult to achieve good and tight placement of the injection packers in open drill holes under such conditions.

For this reason it was decided to drill the boreholes through grouted steel pipes in place. Furthermore, the drilling and injection through the steel pipes was done in advancing steps with a step length of approximately 3 m. Afterwards, the same steel pipe was re-drilled and injected through several times, advancing further forward each time. In this way part of the hole was always situated in improved rock conditions and only a short length was exposed to the collapsing ground.

11.18.4. Results

The first stage injections with rapid hardening micro cement showed a relatively limited grout take of only 100 –150 kg per m borehole before the refusal pressure of 60 bar was reached. Bearing in mind the seepage, which was encountered in the holes, one would have expected a much higher grout take. The reason for the relatively low grout take was joint fillings which consisted of fine grained quartzite, which in turn limited the penetration of the grout. In order to achieve a better penetration into the rock mass, the first stage injection fan was always completed with micro cement. Subsequently, the secondary fan was drilled and injected with liquid colloidal silica with a refusal pressure of 25 bar.

The control of the achieved result was done in two ways. Firstly, the water seepage situation after the injection of the two stages was controlled in the boreholes. Secondly, the result was observed in the tunnel contour after the excavation of the first blasting round, starting from the injection location. In this way, the detailed criteria for termination of the injection was fine-tuned and continuously adjusted.

The result was a literally dry and stable tunnel contour. No excessive breakouts of rock or cave-ins occurred during excavation through the weakness zone.

The time spent on this second crossing of the shear zone was 6 months versus 18 months the first time (resulting in an unusable opening). The average cost of material was EUR 1200 per meter. A similar situation in a tunnel in India required about 10 times this amount for PU used in post-grouting.

12. BASF INJECTION MATERIALS

12.1. The RHEOCEM® range of injection cements

The RHEOCEM micro cements come in three standard types:

1. RHEOCEM 650 pure Portland cement with Blaine value >625 m^2/kg. 100 % of the pareticles are <40 micron.
2. RHEOCEM 800 pure Portland cement with Blaine value >800 m^2/kg. 100 % of the pareticles are <30 micron.
3. RHEOCEM 900 pure Portland cement with Blaine value >900 m^2/kg. 100 % of the pareticles are <20 micron.

These cements have been specifically adapted for use from a tunnel face by giving a very short setting time of about two hours. In the laboratory, a 1:1 water/cement ratio (by weight) at 20°C will give an initial set (measured by the Vicat needle) of 60 to 120 minutes, and final set (defined as 1 mm penetration by the Vicat needle) of 120 to 150 minutes. Under ideal conditions, the open time in the equipment is still one hour, as long as the grout is agitated. The importance of a short setting time has been covered in chapter 2, and the main factor is tunneling economy.

The strength development and final strength is also superior to most other cements and micro cements in the market. When calculating the contractor's cost per meter of tunnel, the time related cost items frequently amount to more than 60%, while cement (when using OPC) regularly cost less than 5%. This is one reason why it is possible to use cement that costs three times as much per kg, and still save money on the total. The work proceeds in less time and with far better results.

The second most important parameter common to all three types is pressure stability, or the low filtration coefficient. It must be emphasized that this property is only maintained when using the prescribed low w/c-ratio of 1.0, and by the use of RHEOBUILD 2000 PF at about 1.5% of the cement weight. This particular admixture gives the combined effect of low viscosity (Marsh Cone flow time of 32 seconds), no segregation and a low filtration coefficient. Thus, penetration is excellent without loss of stability. Practical use in a number of tunnels has demonstrated superior results when other cements have also been tested.

The importance of the right admixture has been demonstrated by test injection in sand-filled Plexiglass tubes. Using RHEOCEM 900, a w/c-ratio of 1.0 and only varying the admixture (28 different products tested), the results gave permeation depths varying from 20 mm to 610 mm. The test setup can be seen in Figure 12.1. The best result was achieved with RHEOBUILD 2000 PF, which is the obvious reason for which this has been selected as the standard admixture. Another reason is that the second best performing admixture only gave about 50% of the best penetration.

Permeation test: 28 different admixtures

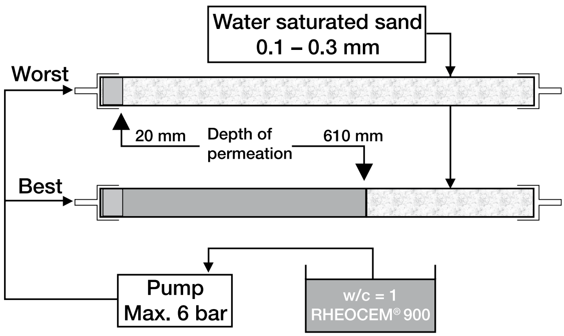


Figure 12.1 Pressure stability and permeation depth dependent on the admixture

The described test is one of the good examples which support the idea that the tendency to focus on the maximum particle size or Blaine value to evaluate permeation properties of different cements is wrong. With RHEOCEM 650 and RHEOBUILD 2000 PF, the permeation depth is frequently almost the same as with RHEOCEM 900. By selecting the wrong admixture with RHEOCEM 900, the permeation depth will typically be poorer than with RHEOCEM 650 and RHEOBUILD 2000 PF.

Proper mixing can only be achieved in a colloidal mixer and the best result is produced by:

- Filling in all the water into the mixer (make sure there are no leaks so the correctly measured water quantity does not change)

- Adding all the cement while running the mixer and mixing for 2 minutes
- Adding the RHEOBUILD 2000 PF and mixing for another minute. Transferring to the agitator

One practical example demonstrating the benefits of using RHEOCEM micro cement versus OPC is the Bekkestua tunnel in Oslo. The contractor Veidekke AS partly used OPC only, partly a combination of OPC and RHEOCEM and partly the micro cement alone. By keeping accurate records of the work progress and the achieved results over the 705 m tunnel length, a very interesting analysis could be carried out by the site manager. Figure 12.2 gives the details of this analysis as received from the contractor.

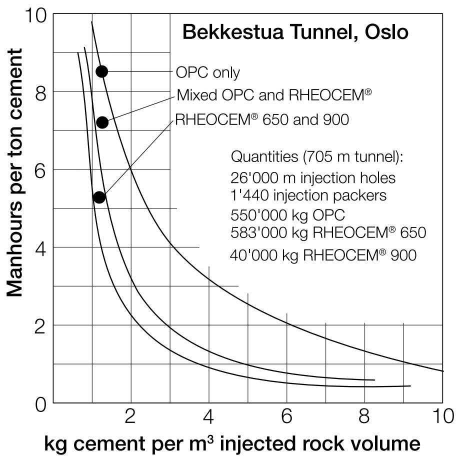


Figure 12.2 Time spent on injection with OPC and micro cement RHEOCEM

As can be seen from the graph, the time spent per kg of injected OPC was typically 2 to 3 times more compared to the time spent using micro cement. It also turned out that the section of the tunnel with the highest recorded residual ingress after tunnel completion had been injected with OPC alone, thus demonstrating the efficiency of the micro cement system.

12.2. Polyurethane grouts

Quick foaming polyurethane grouts are mostly used in underground situations to stop water inrush, but also to fill smaller cavities and to stabilize loose ground.

Comparing it to most other injection materials, its viscosity is rather high. It is obvious that a higher viscosity leads to worse penetration where the permeability is the limiting factor. This is only true for all kinds of materials that do not change their volume during the curing process. For expanding products such as polyurethane foams another aspect has to be considered. These kinds of materials create an additional «inner» pressure, coming from the foaming process. Although the viscosity increases permanently, this inner pressure helps penetration even in small cracks and fissures. Additionally, the shear resistance of rising foam plays an important role for its penetration ability.
Finally, it is left to the applicator to decide on the benefit of a very low viscosity product and therefore exceptional penetration, versus a product with higher viscosity and internal foaming pressure to assist penetration, where the travel distance of the injection product is most likely less, but the targeted ground sealing result the same.

Applying polyurethane grouts is different to cementitious injection, as special pumps are needed and the injection material sticks to everything it comes into contact with. To re-use packers, pipes and valves is possible but not easy, experienced personnel will manage it without difficulty.

Grout and rock/soil temperature have an impact on the reaction speed of polyurethane grouts. If the reaction of the resin is too slow to obtain satisfactory water sealing results because of low temperatures, it is possible to warm up the resin before processing it, thus achieving the required reaction speed.

Polyurethane resins react in any case to moisture and water in the ground, thus affecting penetration and foaming. It must also be mentioned that the foaming factors and statements regarding the foam strength given below refer to free foaming. When injecting into cracks, fissures, sand and ground in general, the space is limited and the foaming factor will always be less than for free foaming; as a consequence the foam strength is higher.

The liquid components can react with proteins of the skin and mucous. Therefore it is mandatory to avoid contact with skin and eyes by using the required personal protective equipment, such as overalls, gloves and safety glasses.

12.2.1. MEYCO® PU grouts for 1 component pumps

MEYCO® MP 355 1K

MEYCO MP 355 1K is a quick foaming material, and it has the advantage of being one-component, so the pumping equipment is quite simple to operate and is reasonably inexpensive. The polyurethane based product is solvent free. It is characterized by its reaction in moist surroundings, and bonds very well to wet surfaces.

Typical cases for the use of MEYCO MP 355 1K are:

- Sealing off flowing water
- Stopping of water inflow before consolidation injection
- Stabilization of jointed rock, coarse sand and gravel

When used for stabilizing highly loosened areas, then the ground mass has to be sufficiently humid to allow thorough curing.

In most of the cases when it is used for running water cut-off, it is beneficial to have as short a reaction time as possible. It is rarely a problem that the reaction time becomes too short, because the foaming continues for a minute or more, and during this time a continued pumping will bring fresh material that creates channels in the foam. The targeted grout quantity will therefore be reached without a premature stop of grout flow.

The foaming reaction requires water (with this product) and is triggered by the first contact with water in the ground. One must be aware that the temperature of the surroundings, the water and the product also have a strong influence on the reaction time. It is therefore normal to carry out site tests to determine the right dosage of accelerator. To give an idea of reaction times, the following has been measured in the laboratory (with 10% accelerator and 10% water):

Initial temperature oC	5	10	15	20
Start of reaction (seconds)	120	60	25	10
End of reaction (seconds)	300	200	110	50
Foam factor (free foaming)	25	25	25	30

Used under wet conditions, the application procedure involves adding the accelerator to the PU at the dosage level established by the pre-testing (2 to 10%), and then mixing it well until the compound of resin and accelerator is free from streaks. It is then injected with a suitable single component pump. Water in the ground will trigger the foaming reaction. It can sometimes be the case that injection takes place under rather dry conditions or in situations where it is unsure if there is sufficient water in the ground. The procedure should then be modified by first pumping water into the rock or soil and then following this by the steps above. This way, one can make sure there is water to trigger the reaction.

When handling accelerated batches of product, one must make sure that the working place is absolutely drip-free. Otherwise, one single drop of water into the mixed product or into the hopper of the pump will start the foaming reaction, and equipment may become clogged up.

MEYCO MP 355 1K is characterized by a high foam factor, which can increase in volume by 25 to 30 times when it is not encumbered, allowing all existing pathways of the water and all open cracks and faults to be fully sealed. As a result of the high foam factor, the stability of the cured foam is relatively low.

The viscosity of the product is about 400 mPa s (20°C).

MEYCO® MP 355 1K DW

MEYCO MP 355 1K DW polyurethane resin is very similar to MEYCO MP 355 1K but especially designed for contact with potable water.

Reaction times are just a few seconds slower, the foam is slightly weaker but less brittle than MEYCO MP 355 1K. The viscosity of the product is about 1000 mPa s (20°C).

All tips and hints given above are also fully applicable to MEYCO MP 355 1K DW.

12.2.2. MEYCO® PU grouts for 2 component pumps

MEYCO® MP 355 A3

For ground water control and running water cut-off, MEYCO MP 355 A3 is an alternative to MEYCO MP 355 1K. This two-component product will be the first choice when using large volumes against large quantities of water. It is characterized by its fast reaction where structural strength or rigidity is required, and it reacts both with and without water: with water, it forms a rigid foam, and without water, a stiff, compact material.

Typical cases for the use of MEYCO MP 355 A3 are:

- Control of high volume water ingress
- Stabilization of fractured rock, sands and gravels, and land-fill materials

This two-component product consists of the A component (polyol) and the B component (isocyanate), which are combined to react rapidly. The foam factor varies, it is one (= no foaming at all) when there is no contact with any water, and it is 3-8 times in free foaming and 15-20 when the accelerators 10 or 25 are used. The chemical reaction of MEYCO MP 355 A3 does not depend on contact with water as all the necessary elements are in the A and B component. This is a significant advantage, as the material will cure under any conditions.

When the two components are mixed together, the following reaction times are observed in the laboratory at 20°C:

Mixed without any presence of water (dry)	
Gel time (open time)	55 s
Setting time (grout becomes hard)	60 s

Mixed with 1 % of water (premixed into the A-component)	
Start of foaming	20 s
End of foaming	60 s

The product may be accelerated by adding different types of accelerator to component A, if necessary:

- Accelerator 10 for a high foam factor of 15-20
- Accelerator 15 for a dense foam with higher mechanical strength, maximum foam factor 7-9
- Accelerator 25 for balanced properties between the 2 types from above, normally the first choice when high amounts of water inrush need to be stopped

The components are delivered ready to use and the two component pump must be set at 1:1 by volume of A and B (this is 1:1.2 by weight). The components are conveyed from the pump to the mixing spiral (static mixer) in separate hoses, and from there onwards in 1 hose through the packer into the ground. When packers contain a static mixing element, separate mixing spirals are not necessary any more.

Due to the exothermic reaction between the two liquid components it is not recommended to inject more than 250 kg of the mixed material into a single borehole. Larger amounts may stay lumped together in bigger cavities and could cause local overheating with a potential risk of smoke development and/or melting and boiling of the resin.

The viscosity of the mixed product is about 300 mPa s (20°C).

MEYCO® MP 355 A3 THIX

MEYCO MP 355 A3 THIX is a special version of MEYCO MP 355 A3 for rapid water stopping under difficult conditions (flowing water). It is characterized by its thixotropic properties, which give increased stability and anti-diluting properties when exposed to high water flows. It is particularly suited to extreme water ingress situations.

The viscosity of the properly mixed product is about 400 mPa s (20°C), The main product difference to MEYCO MP 355 A3 is the very fast viscosity increase, especially at the beginning of the chemical reaction. The properties of the cured product, whether compact or foamed when in contact with water, are similar to those of MEYCO MP 355 A3.

Reaction data under wet conditions:

Initial temperature °C	10	20	30
Start of foaming (seconds)	60	40	20
End of foaming (seconds)	80	60	20
Foam expansion factor (free foaming)	8	15	20

Comparison of initial viscosities of MP 355 A3 and MEYCO MP 355 A3 THIX	**10 seconds after mixing**	**20 seconds after mixing**	**Expansion factor**
MEYCO MP 355 A3 CompA + B, dry	Approx.250 mPas (20°C)	Approx.1 000 mPas (20°C)	1 1 1
MEYCO MP 355 A3 CompA THIX+ B, dry	> 2 000 mPas (20°C)	> 4 000 mPas (20°C)	1 1 1

12.3. Polyurea-silicate grouts

Polyurea-silicate systems are in principle not sensitive to water, which means that they have almost no influence on the foaming factor. This also means that non-foaming products can be used even for injection works under water, where the high strength properties of an un-foamed grout are required. As for polyurethane grouts the surrounding temperature has an influence on the reaction time.

Polyurea-silicate systems are well suited for use in consolidation injection and cavity filling, however they are not designed for water cut-off injection. To process them, the same kind of 2 component pumps can be used as for polyurethane grouts.

From the point of working safety the same standards have to be considered than for polyurethane grouts, as the material also reacts with proteins of the skin and mucous. Therefore it is mandatory to avoid contact with skin and eyes by using the required personal protective equipment, such as overalls, gloves and safety glasses.

12.3.1. Foaming polyurea-silicate grouts

Cavities hamper further operations, and must therefore be filled when rock fall has occurred. Filling such a cavity not only protects the staff from further rock falls, but it also prevents the area from loosening up even more and expanding the cavity.

In order to save time, the material that is used to fill the cavity should cure very quickly. A fast reaction time is not really necessary, yet has possible positive effects on the preparations: the sooner a product reacts, the more the requirements for shutter (formwork) tightness are reduced. If the filling material reacts very fast, the shutter may even be made from planks or mats and prove satisfactory.

Strongly expanding organo-mineral resin foam (polyurea-silicate foam) is well suited for this case. The high foam factor makes it possible to backfill even voluminous overbreak with small amounts of material.

Specially designed products develop less heat when applied in large volumes, thus eliminating the risk of self ignition. It has to be highlighted, that this is superior to all foaming polyurethane grouts!

MEYCO® MP 367 Foam

MEYCO MP 367 Foam is a two-component, solvent-free, self extinguishing polyurea-silicate foam. It is characterized by its high foam factor (25 times) and the zero risk of heat development in large volumes, which would cause self ignition.

Typical cases for the use of MEYCO MP 367 Foam are:

- Void and cavity filling
- Consolidation of fractured rock, sands, gravel and coal, including collapse areas

MEYCO MP 367 Foam expands without coming into contact with water. It does not absorb water and shows good adhesion to wet substrates. It is a very fast reacting material to be applied where foaming speed, flexibility and flame resistance (DIN 4102-1 B2) are required. The product has excellent chemical stability.

The components are delivered ready to use and the two component pump must be set at 1:1 by volume of A and B (this is 100:88 by weight). The components are conveyed from the pump to the mixing spiral (static mixer, length should be around 30cm) in separate hoses, and from there onwards in 1 hose through the packer into the ground or through the outlet hose into the cavity. When packers contain a static mixing element, separate mixing spirals are not necessary any more.
The viscosity of the properly mixed product is about 150 mPa s (20°C).

Reaction characteristics:

Testing temp.	**23°C**
Start of foaming	20s ± 10s
End of foaming	40s ± 15s
Foam expansion factor	about 25

12.3.2. Non - foaming polyurea-silicate grouts

MEYCO® MP 364 Flex

This is the «sister» product of MEYCO MP 367 described above. The main difference is that it does not foam at all, not even under water. The cured product is hard and elastic and bonds well to rock, concrete and coal.

It develops its final high strength values very quickly, which are around 35 MPa compressive strength latest after about 1 day and about 4,5 MPa flexural adhesive strength after 5 minutes.

The main use of this resin is:

- Structural stabilization of broken rock and concrete

- Repair of structures under water / with ground water

One advantage over normal PU products is its low reaction heat and its fire resistance (DIN 4102-1 B2).

The components are delivered ready to use and the two component pump must be set at 1:1 by volume of A and B (this is 100: 79 by weight). The components are conveyed from the pump to the mixing spiral (static mixer, length should be around 30cm) in separate hoses, and from there onwards in 1 hose through the packer into the ground. When packers contain a static mixing element, separate mixing spirals are not necessary any more. The viscosity of the properly mixed product is about 300 mPa s (20°C).

Reaction characteristics:

Testing temp.	23°C
Gel time	90s ± 30s
Setting time	2 min 40s ± 30s
Foam expansion factor	1
Flexural adhesive strength after 24h	3.5 N/mm^2
Border time	<5 min

Border time: Time needed to reach 1 MPa adhesive strength in lab conditions.

12.4. Acrylic grouts

Acrylic resin grouts have a very low viscosity and can therefore be used where maximal penetration is required. The gel formed stays flexible and doesn't shrink provided a minimum humidity is given, which is normally the case for all ground and rock injections. The gel flexibility is particularly beneficial to accommodate movement in the ground or structure.

The open time / gel time of the products dictates the type of pump to be used. Normally it should be a two-component pump made of stainless steel due to the corrosiveness of the grouts. For acrylic systems with open time of more than 15 minutes, 1 component pumps can be used too. As for all chemical injection materials the reaction time is very sensitive to the product and ambient ground temperature.

Acrylic resins are mostly used to permanently stop water in concrete structures and to consolidate ground.

The liquid components can cause irritating effects on skin, eyes and mucous membranes. Repeated skin contact can cause allergic reactions. To minimize the risks of irritation and sensitization, construction workers should wear impermeable overalls, safety glasses, gloves and rubber boots.

MEYCO® MP 301

MEYCO MP 301 contains several acrylic esters and a methacrylamide derivative as an accelerator.

The two-component product is a highly reactive hydrophilic resin that can be used for injection in soil and rock. It contains no toxic components, and can be adjusted by varying the concentration of the mixed product. In contrast to the other acrylic products, MEYCO MP 301 contains neither acrylamide nor formaldehyde, so neither of these substances will be emitted during its application. For ground stabilization MP 301 is particularly well suited, as it gives a strong gel, but it is equally suitable for ground water control. It can be used in temperatures as low as 3°C. To avoid long term shrinkage of the gel, it is important that MEYCO MP 301 remains in a humid environment.

The product is prepared for use by adding a liquid accelerator to component A, while component B is prepared from potable water with up to 5% of the powder hardener. A two-component pump should generally be used. The two pre-made components are usually combined at a 1:1 mixing ratio (by volume). The mixed product will have the following properties:

Density	1.05
pH	6 – 7
Viscosity	< 7 mPa s

The pump must be of stainless steel quality due to the aggressiveness of component B. Containers used during injection should be made of plastic. Typical gel times as found in the laboratory (always test on site):

Comp A	Acc.	Water	Hard.	Start	End
100 %	10 %	100 %	10 %	1’30’’	1’40’’
100 %	8 %	100 %	8 %	1’50’’	2’00’’
100 %	6 %	100 %	6 %	2’20’’	2’40’’
100 %	4 %	100 %	4 %	3’50’’	4’10’’
100 %	3 %	100 %	2 %	5’20’’	5’40’’
100 %	2 %	100 %	2 %	7’30’’	8’10’’
100 %	2 %	100 %	1 %	12’10’’	13’20’’

All primary substances of MEYCO MP 301 are biologically degradable. Acute toxicity towards fish and bacteria is low. Polymerised injection material of MEYCO MP 301 has no ecologically relevant effect.

MEYCO® MP 302

This is an acrylic resin grout very similar to MEYCO MP 301. The main differences are as follows:

- Lower viscosity (at < 3 mPa s)
- Wider range of gel times possible (up to 45 minutes)
- Approved for use in contact with drinking water (NSF approval, www.nsf.org)

When long gel times are used, a one-component pump may be utilized.

Component A			+	**Component B**		
Resin	Acc.	Water		Hard.	Water	**Pot Life**
100%	25%	0%		2,4%	100%	**< 4 min.**
100%	15%	10%		2,4%	100%	**~ 5 min.**
100%	5%	20%		2,4%	100%	**~ 8 min.**
100%	4,5%	20,5%		2,4%	100%	**~ 10 min.**
100%	6%	19%		0,8%	100%	**~ 15 min.**
100%	3,5%	21,5%		0,8%	100%	**~ 20 min.**
100%	2%	23%		0,8%	100%	**~ 30 min.**
100%	1%	24%		0,8%	100%	**~ 40 min.**
100%	0,9%	24,1%		0,8%	100%	**~ 50 min.**

12.5. Colloidal silica (mineral grout)

Colloidal silica is a powerful low viscous injection grout which has only come to use in recent years. Since then it has often been confused with other silicate injection products. Therefore a short description is given here:

Silica is another name for silicon oxides, for example Si02, which can be found in nature in crystalline form, e.g. as quartz (sand), and is the most abundant component of the earth's crust. Colloidal silica is a stable dispersion of silica particles, and can be referred to as "liquid quartz sand". It is a manufactured product and not a by-product from other processes. This gives a very consistent product quality, reproducible performance, and the chemical structure makes the suspension fully stable.

In contrary water glass, otherwise known as sodium silicate, has often been used as injection grout. It has a much higher pH value (up to 13) and has a tendency to leach, thus affecting the pH value of the ground water over time. It also has a significantly higher viscosity. The open time of sodium silicate grouts is strongly dependent on the pH of the surrounding soil. This makes a successful application much more difficult.

Colloidal silica has 2 main benefits: Firstly, it is a grout with exceptional penetration characterises – "it gets where water gets" – and secondly, is completely harmless to human beings and the environment. To enforce this statement it can be mentioned that this technique is also used to purify beer, wine and drinking water.

Penetration is linked to viscosity and particle size. To give an idea of the particle size of colloidal silica, one can refer to the frequently made statement that silica fume particles are like cigarette smoke (see comparison in Table 12.1 below). See also Figure 12.3.1 on next page.

Table 12.1 Comparison of particle size for silica products

Product	**Particle size (μm)**	**Spec. surface (m^2/g)**
Colloidal silica	0.015	80–900
Silica fume	0.2	15–25
Precipitated silica	5	10–15
Crystalline silica (mesh 200)	15	0.4

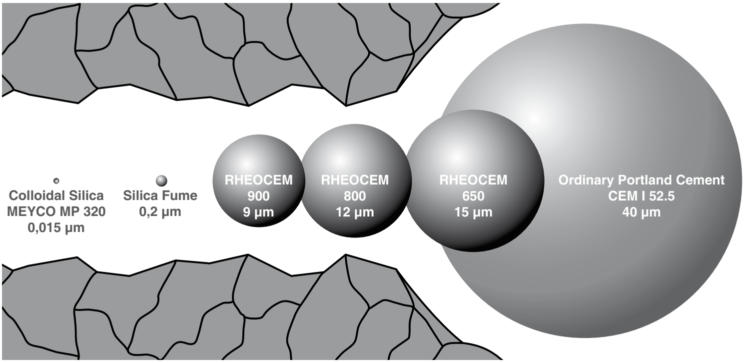


Figure 12.3.1 Injection products particle size

To meet the grouting design expectations, the gel time of the colloidal silica suspension can be adjusted. Therefore an accelerator (component B, which is in fact more of an activator as it only starts the gelling process) is added to the colloidal silica suspension (component A). The gel time depends on the accelerator dosage and the product and ground temperature, thus always requiring some calibration by quick pre-testing.
Figure 12.4 shows the typical relationship between accelerator dosage and gel time.

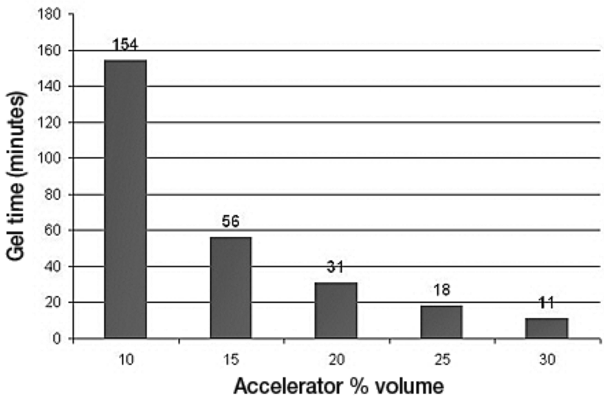


Figure 12.4 Gel time at 8°C versus dosage of accelerator (component B)

The product can be used at temperatures of between +5°C and +40°C.

Due to its very low viscosity, colloidal silica is very easy to pump. Every pump that is suited for pumping water can be used. This gives plenty freedom in the choice of pumps. For small works, a small pump can be used, and for big tunnel jobs normal grout plants are suitable. Colloidal silica and the accelerator mix together very easily, 20 – 30 seconds is enough.

Normally, one-component injection pumps as used for cement will be fully satisfactory. The two components then have to be pre-mixed in batches with the chosen volume ratio and gel time, before feeding the mix to the pump. It is important to keep the mixer running when adding the accelerator to the colloidal silica. This is necessary to avoid lumps in the mix as a result of “local overdosing” of accelerator. Fresh batches should not be mixed into a remaining volume of a batch at the end of its open time. This will shorten the gel time of the fresh batch substantially and may cause unexpected problems.

It is also possible to use 2 component pumps. To have control of the open time, flow meters are needed on both accelerator and product line. An advantage of using a 2 component system is that the accelerator dosage can be adjusted during injection. To secure a good mixing of colloidal silica and accelerator, they have to be mixed in an in-line mixing spiral before entering the strata.

If the colloidal silica gels inside the injection equipment, this will normally not cause any big problems. The gel is weak, and it is easy for a grout pump to pump the gel out without the risk of clogging the equipment and injection hose. Cleaning the equipment is easily done using water. None of the components contain toxic, aggressive or harmful substances.

The gelling behaviour of MEYCO® MP colloidal silica products is very favourable because the low viscosity is maintained until the preset gel time, at which point the viscosity increases rapidly. This is well demonstrated in the measurements presented in Figure 12.5.

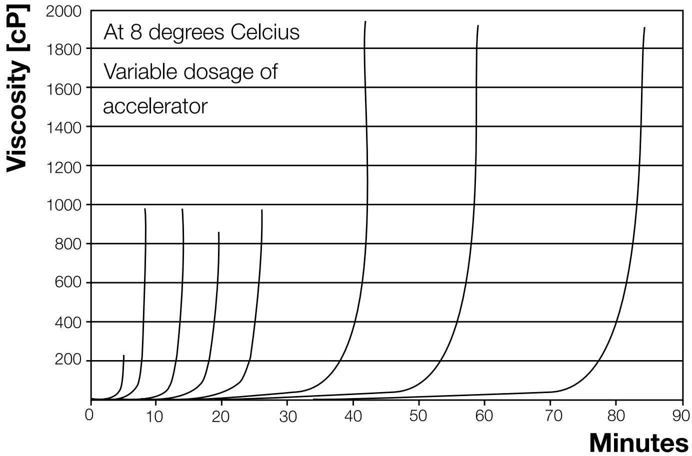


Figure 12.5 Viscosity development of MEYCO MP colloidal silica

The strength of the gel is higher than the traditional silicate gel products, and MEYCO MP colloidal silica products show zero syneresis and no shrinkage. This creates very good water tightness, and the ground strengthening effect is also noticeable in loose soils, running sand and in very broken rock. Like any other gel product, it will dry out and shrink if exposed for a long time to atmospheric conditions at lower relative humidity. However, this is not a problem underground in soil and rock, when injected to seal off ground water. Compared to traditional silicate grouts, the chemical stability is considerably improved. The strength development has been measured to continue for more than 7 years, and there is no reason to expect this to stop or reverse. Theoretical studies carried out by Prof. Yonekura (Japan) indicate the strength will never stop increasing.

The gel itself has a compressive strength of 0.1 MPa 1 day after injection, while injected silty sand would reach 0.5 to 1.0 MPa 2 to 3 weeks after injection. The strength is influenced by the type of sand / silt. The following general rule applies: The finer the material, the higher the strength. Compressive strength is generally not a parameter of importance, but it illustrates the ballpark properties of the injected soil when stability is an issue.

This product represents an entirely new opportunity in rock and soil injection, primarily because of its unique combination of positive properties:

- Very low viscosity
- Environmentally friendly
- Easy to handle
- Improved working safety
- Long open time and controlled gelling
- Excellent chemical stability and durability

Main areas of use:

- Pre-injection grouting for tunneling projects where penetration is of utmost importance
- Ground injection where environmentally safe grouts are needed
- Stabilization of dense sands where cementitious injections have come to their limit

MEYCO® MP 320

MEYCO MP 320 is a water suspension of nanometric silica particles. The water dispersion contains discrete, non-aggregated spherical particles of 100% amorphous silicon dioxide in suspension. Furthermore, the product can be considered a permanent sealant.
The viscosity of the mixed product is about 5 mPa.s

MEYCO® MP 325

MEYCO MP 325 is as MEYCO MP 320 a water suspension of nanometric silica particles.

Comparing the 2 products, MEYCO MP 325 has:

- a lower solid content but finer particles
- a higher accelerator consumption
- a shorter shelf life
- a higher theoretical shrinkage in a dry environment

but applied properly it allows comparable injection results to MEYCO MP 320.

The viscosity of the mixed product is about 10 mPa.s

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